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Indian Standard

CODE OF PRACTICE FOR CALCULATION OF SETTLEMENTS OF FOUNDATIONS

PART I SHALLOW FOUNDATIONS SUBJECTED TO SYMMETRICAL STATIC VERTICAL LOADS

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Indian Standard
CODE OF PRACTICE FOR CALCULATION OF SETTLEMENTS OF FOUNDATIONS
PART I SHALLOW FOUNDATIONS SUBJECTED TO SYMMETRICAL STATIC VERTICAL LOADS

0. FOREWORD

0.1 This Indian Standard (Part I) was adopted by the Indian Standards Institution on 21 February 1976, after the draft finalized by the Foundation Engineering Sectional Committee had been approved by the Civil Engineering Division Council.

0.2 Settlement may be the result of one or combinations of the following causes:

a) Static loading;
b) Deterioration of foundation;
c) Mining subsidence; and
d) Shrinkage of soil, vibration, subsidence due to underground erosion and other causes.

0.2.1 Catastrophic settlement may occur, if the static load is excessive. When the load is not excessive, the resulting settlement may consist of the following components:

a) Elastic deformation or immediate settlement of foundation soil,
b) Primary consolidation of foundation soil resulting from the expulsion of pore water,
c) Secondary compression of foundation soil, and
d) Creep of the foundation soil.

0.3 If a structure settles uniformly, it will not theoretically suffer damage, irrespective of the amount of settlement. But, the underground utility lines may be damaged due to excessive settlement of the structure. In practice, settlement is generally non-uniform. Such non-uniform settlements induce secondary stresses in the structures. Depending upon the permissible extent of these secondary stresses, the settlements have to be limited. Alternatively, if the estimated settlements exceed the allowable limits, the foundation dimensions or the design may have to be suitably modified. Therefore, this code is prepared to provide a common basis, to the extent possible, for the estimation of the settlement of shallow foundations subjected to symmetrical static vertical loading.

0.4 A settlement calculation involves many simplifying assumptions as detailed in **4.1**. In the present state of knowledge, the settlement computations at best estimate the most probable magnitude of settlement.

0.5 In the formulation of this standard due weightage has been given to international co-ordination among the standards and practices prevailing in different countries in addition to relating it to the practices in the field in this country.

1. SCOPE

1.1 This standard (Part I) provides simple methods for the estimation of immediate and primary consolidation settlements of shallow foundations under symmetrical static vertical loads. Procedures for computing time rate of settlement are also given.

1.2 This standard does not deal with catastrophic settlement as the foundations are expected to be loaded only up to the safe bearing capacity. Analytical methods for the estimation of settlements due to deterioration of foundations, mining and other causes are not available and, therefore, are not dealt with. Satisfactory theoretical methods are not available for the estimation of secondary compression. However, it is known that in organic clays and plastic silts, the secondary compression may be important and, therefore, should be taken into account. In such situations, any method considered suitable for the type of soil met with may be adopted by the designer.

2. TERMINOLOGY

2.0 For the purpose of this standard, tne following definitions shall apply.

2.1 Coefficient of Compressibility — The secant slope, for a given pressure increment, of the effective pressure-void ratio curve.

2.2 Coefficient of Consolidation (c_v)— A coefficient utilized in the theory of consolidation, containing the physical constants of a soil affecting its rate of volume change:

$$c_v = \frac{k(1+e)}{a_v \gamma_w}$$

where

k = coefficient of permeability,

e = void ratio,

a_v = coefficient of compressibility, and

γ_w = unit weight of water.

2.3 Coefficient of Volume Compressibility — The compression of a soil layer per unit of original thickness due to a given unit increase in pressure. It is numerically equal to the coefficient of compressibility divided by one plus the original void ratio, that is

$$\frac{a_v}{1+e}$$

2.4 Compression Index — The slope of the linear portion of the pressure-void ratio curve on a semi-log plot, with pressure on the log scale.

2.5 Creep

a) Slow movement of soil and rock waste down slopes usually imperceptible except to observations of long duration.

b) The time dependent deformation behaviour of soil under constant compressive stress.

2.6 Degree of Consolidation (Percent Consolidation) — The ratio, expressed as a percentage of the amount of consolidation at a given time, within a soil mass to the total amount of consolidation obtainable under a given stress condition.

2.7 Effective Stress (Intergranular Pressure) — The average normal force per unit area transmitted from grain to grain of a soil mass. It is the stress which to a large extent controls the mechanical behaviour of a soil.

2.8 Elastic Deformation (Immediate Settlement) — It is that part of the settlement of a structure that takes place immediately on application of the load.

2.9 Immediate Settlement — *See* **2.8**.

2.10 Intergranular Pressure — *See* **2.7**.

2.11 Liquid Limit — The water content, expressed as a percentage of the weight of the oven dry soil, at the boundary between liquid and plastic states of consistency of soil.

2.12 Normally Consolidated Clay — A soil deposit that has never been subjected to an effective pressure greater than the existing effective pressure.

2.13 Overconsolidated Soil Deposit — A soil deposit that has been subjected to an effective pressure greater than the present effective pressure.

2.14 Pore Pressure — Stress transmitted through the pore water.

2.15 Primary Consolidation — The reduction in volume of a soil mass caused by the application of a sustained load to the mass and due principally to a squeezing out of water from the void spaces of the mass and accompanied by a transfer of the load from the soil water to the soil solids.

2.16 Secondary Compression — The reduction in volume of a soil mass caused by the application of a sustained load to the mass and due principally to the adjustment of the internal structure of the soil mass after most of the load has been transferred from the soil water to the soil solids.

2.17 Sensitive Clay — A clay which exhibits sensitivity.

2.18 Sensitivity — The ratio of the unconfined compressive strength of an undisturbed specimen of the soil to the unconfined compressive strength of specimen of the same soil after remoulding at unaltered water content. The effect of remoulding on the consistency of a cohesive soil.

2.19 Shallow Foundation — A foundation whose width is greater than its depth. The shearing resistance of soil above the base level of the foundation is neglected.

2.20 Static Cone Resistance — Force required to produce a given penetration into soil of a standard static cone.

2.21 Time Factor — Dimensionless factor, utilized in the theory of consolidation, containing the physical constants of a soil stratum influencing its time-rate of consolidation, expressed as follows:

$$T = \frac{k\,(1+e)\,t}{a_v\,\gamma_w\,H^2} = \frac{c_v\,t}{H^2}$$

where

t = elapsed time that the stratum has been consolidated; and

H = maximum distance that water must travel in order to reach a drainage boundary; it will be equal to the thickness of layer in the case of one way drainage and half the thickness of layer in the case of two way drainage.

2.22 Void Ratio — The ratio of the volume of void space to the volume of solid particles in a given soil mass.

2.23 Water Table — Elevation at which the pressure in the ground water is zero with respect to the atmospheric pressure.

3. SYMBOLS

3.0 For the purpose of this standard and unless otherwise defined in the text, the following symbols shall have the meaning indicated against each:

a_v = Coefficient of compressibility, m^2/kg

B = Width of footing, m

c = Constant of compressibility

C_c = Compression index

C_{kd} = Static cone resistance, kg/cm^2

c_v = Coefficient of consolidation, m^2/year
D = Depth of footing, m
d = Depth of water table below foundation, m
E = Modulus of elasticity, kg/cm^2
E_h = Young's modulus in the horizontal direction, kg/cm^2
E_v = Young's modulus in the vertical direction, kg/cm^2
e = Void ratio
e_0 = Initial void ratio at mid-height of layer
H_t = Thickness of soil layer, m
H = Thickness of compressible stratum measured from foundation level to a point where induced stress is small for drainage in one direction; half the thickness of compressible stratum below the foundation for drainage in two directions, m
I = Influence factor for immediate settlement
I_B = Influence value for stress
k = Coefficient of permeability, m/year
L = Length of footing, m
m' = Frolich concentration factor
m_v = Coefficient of volume compressibility, cm^2/kg
P = Concentrated load, kg
p = Foundation pressure, kg/cm^2
$\bar{p}$ = Effective pressure, kg/cm^2
$\bar{p}_c$ = Maximum intergranular pressure, kg/cm^2
$\bar{p}_0$ = Initial effective pressure at mid-height of layer, kg/cm^2
R = Radial ordinate in 3D case (*see* Fig. 15)
S_1 = Settlement of a footing of 30×30 cm, m
S_c = Primary consolidation settlement, m
S_f = Final settlement, m
S_{fd} = Settlement corrected for effect of depth of foundation, m
S_i = Immediate settlement, m
S_{oed} = Settlement computed from one dimensional consolidation test, m
S_t = Total settlement at time t, m
T = Time factor
t = Elapsed time, year
U = Degree of consolidation
w_L = Liquid limit
z = Vertical ordinate
β = Angular coordinate in 3D case (*see* Fig. 15)

Δp = Pressure increment, kg/cm^2
μ = Poisson's ratio
λ = A factor related to pore pressure parameter A and the dimensions of loaded area (*see* Table 1)
η = A factor used in Westergaard theory
σ_z = Vertical stress, kg/cm^2

4. GENERAL CONSIDERATIONS

4.1 Requirements, Assumptions and Limitations

4.1.1 The following information is necessary for a satisfactory estimate of the settlement of foundations:

a) Details of soil layers including the position of water table.
b) The effective stress-void ratio relationship of the soil in each layer.
c) State of stresses in the soil medium before the construction of the structure and the extent of overconsolidation of the foundation soil.

4.1.2 The following assumptions are made in settlement analysis:

a) The total stresses induced in the soil by the construction of the structure are not changed by the settlement.
b) Induced stresses on soil layers due to imposed loads can be estimated.
c) The load transmitted by the structure to the foundation is static and vertical.

4.1.3 The thickness and location of compressible layers may be estimated with reasonable accuracy. But often the soil properties are not accurately known due to sample disturbance. Further, the properties may vary within each layer and average values may have to be worked out.

4.1.4 The settlement computation is highly sensitive to the estimation of the effective stresses and pore pressures existing before loading and the stress history of the soil layer in question.

The methods suggested in this standard for the determination of these parameters and evaluation of the extent of overconsolidation are generally expected to yield satisfactory results.

4.1.5 Unequal settlement may cause a redistribution of loads on columns. Therefore, settlement may cause changes in the loads acting on the foundation. To take into account the effect of settlement on redistribution of loads, a trial and error procedure may be adopted. In this procedure, the settlements are first computed assuming that the load distribution is independent of the settlements. Then, the differences in settlement between different parts of the structure are worked out and also new trial values of load distribution are computed. By trial and error, the settlements at different parts of the structure are made compatible with the structural loads which caused them. However, in clays, as the settlements often occur over long periods, the readjustment of column loads due to creep in structural materials are often not taken note of.

4.1.6 The soil layers experience stresses due to the imposed loads. The distribution of pressure on the soil at the foundation level, that is, contact pressure distribution, depends upon several factors such as the rigidity of the soil, rigidity of foundation etc; but it is customary to assume that the soil reaction is planar and compute the stresses induced in the compressible layers from simple formulae based on theory of elasticity such as Boussinesq's and Westergaard's equations. The use of these equations for computation of stresses in layered and non-uniform deposits is questionable. The extent of errors introduced are also not known. However, it is generally believed that the application of the methods suggested in this standard leads to errors on the safe side.

4.2 Soil Profile

4.2.1 For calculation purposes, the soil profile may be simplified into one or more layers depending upon the extent of uniformity. For each layer, the average compressibility is estimated. The settlement at any point is computed as the sum of the settlements of all the layers below this point, which are affected by the superimposed loads.

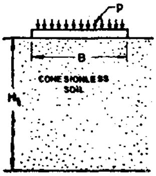
4.2.2 The following are the possible types of soil formations:

a) A deposit of cohesionless soil resting on rock,
b) A thin clay layer sandwiched between cohesionless soil layers or between a cohesionless soil layer at top and rock at bottom,
c) A thin clay layer extending to ground surface and resting on cohesionless soil layer or rock,
d) A thick clay layer resting on cohesionless layer or rock,
e) A deposit of several regular soil layers, and
f) An erratic soil deposit.

These soil formations are shown in Fig. 1 to 6.

4.2.3 Where there are both cohesionless and cohesive soil layers, unless the cohesive layer is thin or the cohesive soil is stiff or the cohesionless soil is very loose, the settlement contribution by the cohesionless soil layers will be small compared to that due to the cohesive soil layers; and the former may be neglected. In the exceptional cases cited above, the settlement due to all the layers have to be estimated. Rock is incompressible compared to soil and therefore, the settlement of the rock stratum is neglected.

4.2.4 The method of computation for cohesionless deposit differs from that for a cohesive deposit. The settlement of a thin clay layer shown in Fig. 2 is one dimensional, whereas in a thin clay layer shown in Fig. 3 or a thick clay layer, the settlement depends upon lateral deformation. Therefore, the different types of soil formations require different methods of settlement computation. These different methods are stipulated in subsequent clauses.



No limitation for H_t

FIG. 1 DEPOSIT OF COHESIONLESS SOIL RESTING ON ROCK

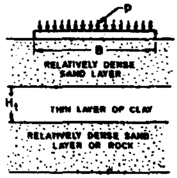
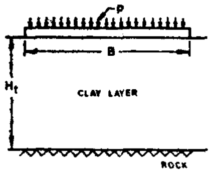
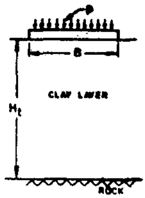


FIG. 2 THIN CLAY LAYER SANDWICHED BETWEEN COHESIONLESS SOIL LAYERS OR BETWEEN A COHESIONLESS SOIL LAYER AT TOP AND ROCK AT BOTTOM



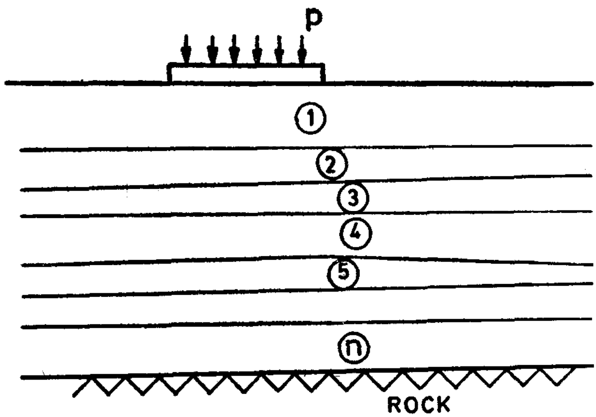
$B > H_t$

FIG. 3 THIN CLAY LAYER EXTENDING TO GROUP SURFACE AND RESTING ON COHESIONLESS SOIL LAYER OR ROCK



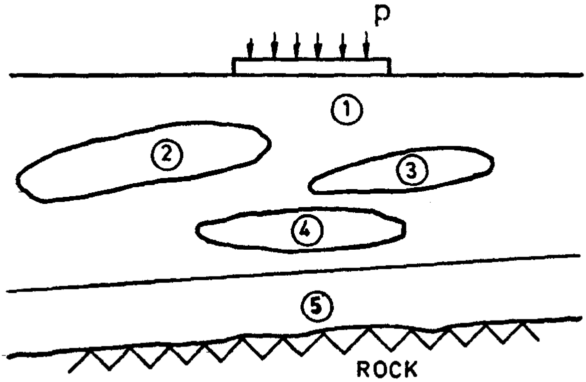
$B < H_t$

FIG. 4 THICK CLAY LAYER RESTING ON COHESIONLESS LAYER OR ROCK



1, 2, 3.......n are independent layers

FIG. 5 DEPOSIT OF SEVERAL REGULAR SOIL LAYERS



1, 2, 3......are independent layers

FIG. 6 ERRATIC SOIL DEPOSIT

5. STEPS INVOLVED IN SETTLEMENT COMPUTATIONS

5.1 The following are the necessary steps in settlement analysis:

a) Collection of relevant information,
b) Determination of a subsoil profile,
c) Stress analysis,
d) Estimation of settlements, and
e) Estimation of time rate of settlements.

6. COLLECTION OF RELEVANT INFORMATION

6.1 The following details pertaining to the proposed structure are required for a satisfactory estimation of settlements:

a) Site plan showing the location of proposed as well as neighbouring structures,
b) Building plan giving the detailed layout of load bearing walls and columns and the dead and live loads to be transmitted to the foundation,
c) Other relevant details of the structure, such as rigidity of structure, and
d) A review of the performance of structures, if any, in the locality and collection of data from actual settlement observation on structures in the locality.

6.2 As the settlement behaviour of the soil profile is affected by the drainage and possible flooding conditions adjacent to the site and presence or absence of fast growing and water seeking trees, these informations may also be collected.

7. DETERMINATION OF SUB-SOIL PROFILE

7.1 It is required that a sufficient number of borings be taken in accordance with IS : 1892-1962* to indicate the limits of various underground strata and to furnish the location of water table and water bearing strata. Generally, it will be sufficient, if two exploratory holes are located diagonally on opposite corners, unless the proposed structure is small, in which case one bore hole at the centre may be sufficient. For large structures, additional bore holes may be driven suitably.

7.2 A plot of the borings is likely to show some irregularities in the various strata. However, in favourable cases, all borings may be sufficiently alike to allow the choosing of an idealised profile, which differs only slightly from any individual borings and which is a close representation of average strata characteristics.

7.3 Adequate boring data and good judgement in the interpretation of the data are prime requisites in the calculation of settlements. In the case of cohesionless soils, the data should include the results of standard penetration tests (*see* IS : 2131-1963†). In the case of cohesive soils, the data should include consolidation test results on undisturbed samples [*see* IS : 2720 (Part XV)-1965‡]. Accuracy of settlement calculation improves with increasing number of consolidation tests on undisturbed samples. The number of samples to be tested depends on the extent of uniformity of soil deposit and the significance of the proposed structure. In general, in the case of clay layers, for each one of the clay layers within the zone of stress influence at least one undisturbed sample should be tested for consolidation characteristics. In the case of thick clay layers, consolidation test should be done on samples collected at 2 m or lesser intervals.

8. STRESS ANALYSIS

8.1 Initial Pore Pressure and Effective Stress

8.1.1 The total vertical pressure at any depth below ground surface is dependent only on the weight of the overlying material. The values of natural unit weight obtained for the samples at different depths, should be used to compute the pressures. To obtain initial effective pressure, neutral pressure values should be subtracted from the total pressure. The

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*Code of practice for site investigation for foundations.

†Method of standard penetration test for soils.

‡Methods of test for soils: Part XV Determination of consolidation properties.

possible major types of preloading conditions that can exist are the following (*see* Fig. 7):

a) Simple static case,
b) Residual hydrostatic case,
c) Artesian case, and
d) Overconsolidated case.

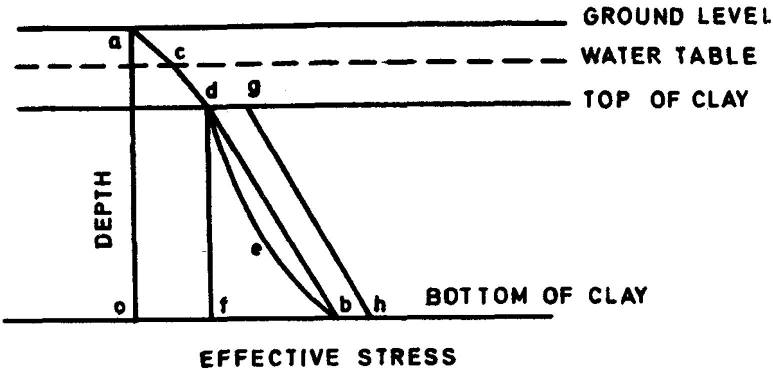


FIG. 7 PRELOADING CONDITIONS

8.1.2 In simple static case and in the overconsolidated case, the neutral pressure at any depth is equal to the unit weight of water multiplied by the depth below the free water surface. In residual hydrostatic case, a condition of partial consolidation under the overburden exists, if part of the overburden has been recently placed as for example in made up lands and delta deposits. In this case, the neutral pressure is greater than that in the previous case, since it includes hydrostatic excess pressure. If allowed sufficient time, this case would merge with the static case. The artesian case is that in which there is upward percolation of water through the clay layer due to natural or artificial causes. In this case, in addition to the hydrostatic pressure a seepage pressure acts upwards and reduces the effective pressure in the soil. In the pre-compressed or overconsolidated case, the clay might have been subjected to a higher effective pressure in the past than exists at present. This may be due to the water table having been lower in the past than at present or due to the erosion or removal of some depth of material at the top. The identification of one of the four cases listed above for the given site conditions is essential for the satisfactory evaluation of magnitude of settlements. The detailed procedure and the implications are described in Appendix A.

8.2 Pressure Increment

8.2.1 The pressure increment is defined as the difference between the initial intergranular pressure as existed in the field prior to application of load and the final intergranular pressure after application of load. The following may contribute to the pressure increment:

a) Pressure transmitted to the clay by construction of buildings or by other imposed loads,

b) Residual hydrostatic excess pressures,

c) Pressure changes caused by changes in the elevation of the water table above the compressible stratum, and

d) Pressure changes caused by changes in the artesian pressure below the compressible stratum.

8.2.2 The pressure increment due to imposed loads shall be determined as detailed in **8.3**. The residual hydrostatic excess pressure should be estimated by the methods given in Appendix A. The pressure changes caused by changes in water table elevations or artesian pressures may be estimated by field observations of ground water elevations or by anticipated fluctuations in the water table or artesian pressures due to natural or man-made causes.

8.3 Selection of Stress Distribution Theories to Compute the Pressure Increment due to Imposed Loads

8.3.1 The pressure increment induced by the imposed loads are usually determined using the isotropic, homogeneous and elastic half space solutions due to Boussinesq.

8.3.2 Normally consolidated clays may be considered isotropic for this purpose and therefore, Boussinesq theory is applicable.

8.3.3 Overconsolidated and laminated clays can be expected to exhibit marked anisotropy. For such clays, Westergaard's results may be used.

8.3.4 Sand deposits exhibit marked decrease in compressibility with depth. Fröhlich's solutions may be used for such deposits.

8.3.5 For variable deposits, the Boussinesq's solutions may be used.

8.3.6 The details of the procedure for estimating the pressure increment due to imposed loads and the relevant charts are given in Appendix B.

9. ESTIMATION OF TOTAL AND DIFFERENTIAL SETTLEMENTS

9.1 Estimation of Settlements of Foundation on Cohesionless Soils

9.1.1 Settlements of structures on cohesionless soils such as sand take place immediately as the foundation loading is imposed on them. Because of the difficulty of sampling these soils, there are no practicable laboratory procedures for determining their compressibility characteristics. Consequently, settlement of cohesionless soil deposits may be estimated by a semi-empirical method based on the results of static cone or dynamic penetration test or plate load tests.

9.1.2 *Method Based on Static Cone Penetration Test* — Static cone penetration test should be performed in accordance with IS : 4968 (Part III)-1971*. A curve showing the relationship between depth and static cone penetration resistance (*see* Fig. 8) should be prepared. It is broken into several parts, each part having approximately same value. The average cone resistance of each layer is taken for calculating the constant of compressibility. The settlement of each layer within the stressed zone due to the foundation loading, should be separately calculated using equation (1) and the results added together to give the total settlement.

$$S_f = 2{\cdot}303 \frac{H_t}{C} \log_{10} \left[\frac{\bar{p}_o + \Delta p}{\bar{p}_o}\right] \quad .. \quad .. \quad .. \quad (1)$$

$$c = \frac{3}{2} \frac{C_{kd}}{\bar{p}_o} \quad .. \quad .. \quad .. \quad (2)$$

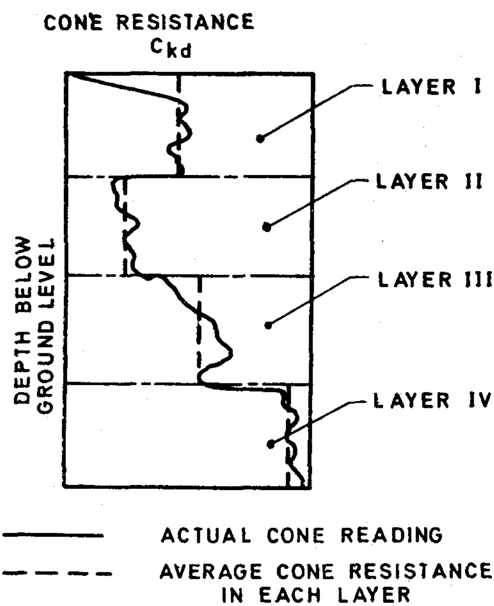


FIG. 8 STATIC CONE PENETRATION RESISTANCE DIAGRAM

*Method for subsurface sounding for soils, Part III Static cone penetration test.

9.1.3 *Method Based on Plate Load Test* — Plate load test should be performed at the proposed foundation level and the settlement of a 30 cm square plate under the design intensity of loading on the foundation estimated (*see* IS : 1888-1971*). Then the total settlement of the proposed foundation is given by:

$$S_f = S_1 \left(\frac{2H}{B+0{\cdot}30}\right)^2 \quad \ldots \quad (3)$$

where

S_f = final settlement in metres for a given load of p per unit area of a footing of width B, and

S_1 = settlement in metre of a footing with a loaded area of 30×30 cm under a given load p per unit area.

9.1.4 *Method Based on Dynamic Penetration Test* — Settlement of a footing of width B under unit intensity of pressure resting on dry cohesionless deposit with known standard penetration resistance value N, (determined according to IS : 2131-1963†), may be read from Fig. 9. The settlement under any other pressure may be computed by assuming that the settlement is proportional to the intensity of pressure. If the water table is at a shallow depth, the settlement read from Fig. 9 shall be multiplied by the correction factor W' read from the inset in the same figure.

NOTE — The standard penetration value obtained in the test shall be corrected for overburden and for fine sand below water table as detailed in IS : 6403-1971‡.

9.2 Estimation of Total Settlement of Foundation on Cohesive Soils

9.2.1 In the case of clay layers, the total settlement should be computed from

$$S_f = S_1 + S_c \quad \ldots \quad (4)$$

Procedures for estimation of immediate and consolidation settlements differ for different types of soil profiles and are explained in **9.2.2** and **9.2.3**.

9.2.2 *Clay Layer Sandwiched between Cohesionless Soil Layers or between a Cohesionless Soil Layer at Top and Rock at Bottom*

9.2.2.1 For situations shown in Fig. 2 it may be assumed that:

$$S_f = 0$$

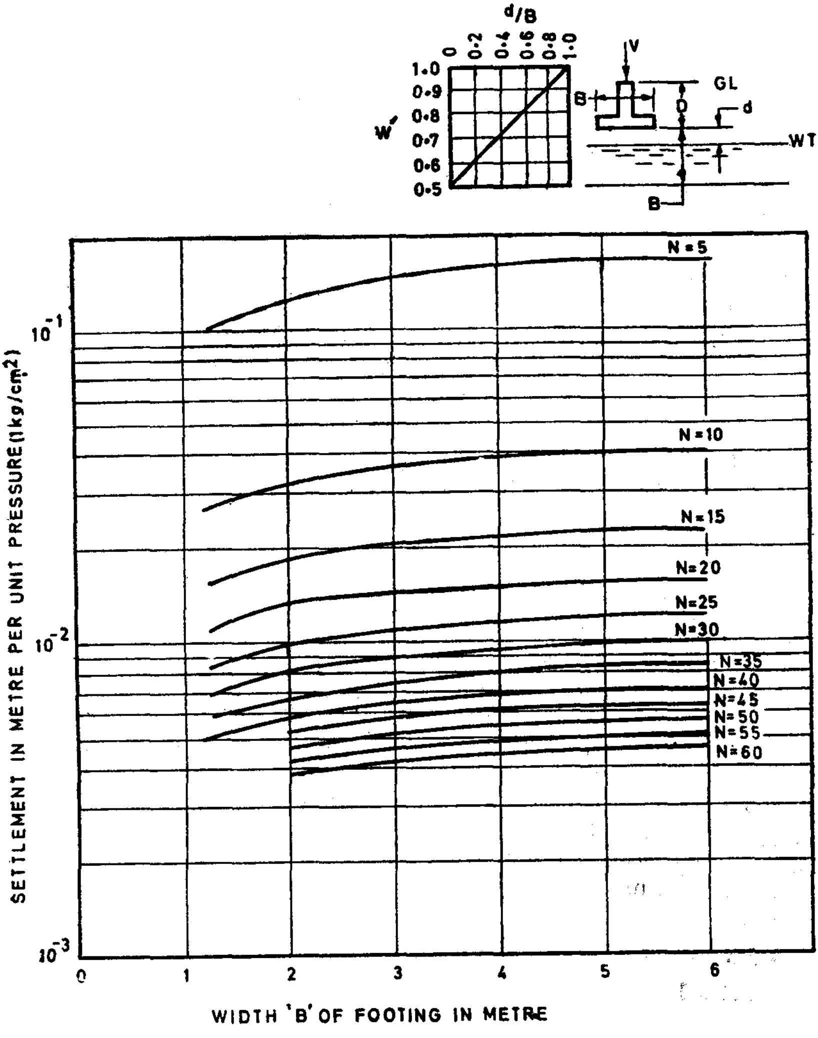
and $$S_c = S_{oed} \quad \ldots \quad (5)$$

The details of the computations of S_{oed} depends upon the preloading conditions. The details are given in **9.2.2.2** to **9.2.2.4**.

*Method of load tests on soils (*first revision*).

†Method of standard penetration test for soils.

‡Code of practice for determination of allowable bearing pressure on shallow foundations.



GL = Ground level. WT = Water table.

FIG. 9 SETTLEMENT PER UNIT PRESSURE FROM STANDARD PENETRATION RESISTANCE

9.2.2.2 If the clay is not precompressed, that is, in a simple static or residual hydrostatic excess or artesian case, the settlement may be calculated by:

$$S_f = S_{oed} = \frac{H_t}{1+e_0} C_c \log_{10} \left[\frac{\bar{p}_0 + \Delta p}{\bar{p}_0} \right] \quad .. \quad .. \quad .. \quad (6)$$

The initial effective vertical stress and pressure increment should be obtained as detailed in **8.1** and used in equation (6) to estimate the probable settlements. The compression index should be determined from the consolidation test data (*see* Appendix C). In preliminary investigations the compression index may also be estimated from empirical formulae, provided, the clay is not extremely sensitive or highly organic. The following two empirical formulae are recommended:

$$C_c = 0{\cdot}009\ (w_L - 10) \quad .. \quad .. \quad .. \quad (7)$$

$$C_c = 0{\cdot}30\ (e_0 - 0{\cdot}27) \quad .. \quad .. \quad .. \quad (8)$$

9.2.2.3 In the case of precompressed clays, the compressibility will be considerably lower than that of a normally consolidated clay. The settlement in this case, may be computed from:

$$S_f = \Delta p\ m_v\ H \quad .. \quad .. \quad .. \quad .. \quad (9)$$

The coefficient of volume compressibility is generally obtained from consolidation test result for the range of loading. But, because of overconsolidation, this value of volume compressibility differs from the field value. A somewhat reliable method of estimating the field value of m_v is given in Appendix C. This value shall be used in equation (9) to obtain the settlement. If the clay is heavily overconsolidated, then, it may not be possible to adopt this procedure. But, in such cases, it may not be necessary to compute the settlement.

9.2.2.4 If the clay layer shows marked change in compressibility with depth, the clay layer shall be divided into a number of sublayers and the settlement in each sublayer shall be computed separately. Then, the total settlement is given by the sum of the settlements of each individual sublayer.

9.2.3 *Clay Layer Resting on Cohesionless Soil Layer or Rock*

9.2.3.1 For cases illustrated in Fig. 3 and 4,

$$S_c = \lambda S_{oed} \quad .. \quad .. \quad .. \quad .. \quad (10)$$

where

λ = A factor related to the pore pressure parameter A and the ratio H_t/B, and read from Fig. 10. In the absence of data regarding the pore pressure parameter A, it may be sufficient to take the value of λ from Table 1.

S_{oed} shall be estimated as in **9.2.2**.

TABLE 1 VALUES OF λ

TYPE OF CLAY	λ
(1)	(2)
Very sensitive clays (soft alluvial, estuarine and marine clays)	1·0 to 1·2
Normally consolidated clays	0·7 to 1·0
Overconsolidated clays	0·5 to 0·7
Heavily overconsolidated clays	0·2 to 0·5

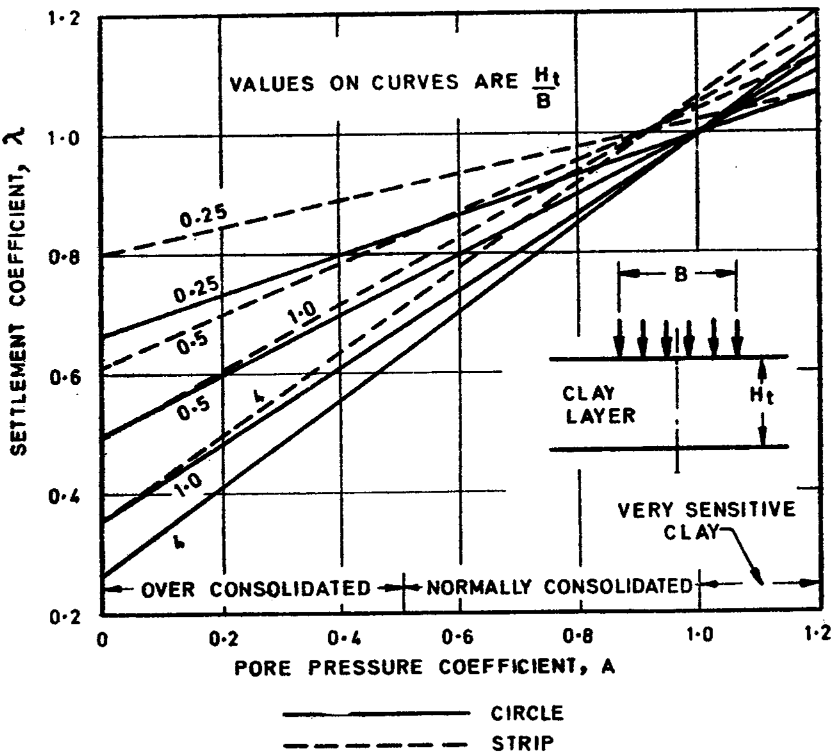


FIG. 10 SETTLEMENT COEFFICIENTS FOR CIRCULAR AND STRIP FOOTINGS

9.2.3.2 The immediate settlement beneath the centre or corner of a flexible loaded area is given by:

$$S_i = pB \frac{(1-\mu^2)}{E} I \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad (11)$$

where

μ = Poisson's ratio = 0·5 for clay, and

I = Influence factor [depends on length (L) to breadth (B) ratio of the footing].

Values of E shall be determined from the stress strain curve obtained from triaxial consolidated undrained test. The consolidation pressure adopted in triaxial consolidation test should be equal to the effective pressure at the depth from which the sample has been taken. The values of I may be determined from Fig. 11 for clay layers with various H_t/B ratio and from Table 2 for clay layers of semi-infinite extent.

TABLE 2 VALUES OF *I* FOR CLAY LAYERS OF SEMI-INFINITE EXTENT

Shape	Influence Factor (I)		
	Centre	Corner	Average
(1)	(2)	(3)	(4)
Circle	1·00	0·64 (edge)	0·85
Square	1·12	0·56	0·95
Rectangle:			
L/B = 1·5	1·36	0·68	1·20
2	1·53	0·77	1·31
5	2·10	1·05	1·83
10	2·52	1·26	2·25
100	3·38	1·69	2·96

9.3 Several Regular Soil Layers — If the soil deposit consists of several regular soil layers, the settlement of each layer below the foundation should be computed and summed to obtain the total settlement. The settlement contribution by cohesionless soil layers should be estimated by the methods in **9.1**; similarly the settlement contribution by cohesive soil layers should be estimated by the methods in **9.2**.

9.4 Erratic Soil Deposit

9.4.1 In variable erratic soil deposits, if the variation occurs over distances greater than half the width of foundation, settlement analysis should be based on the worst and the best conditions. That is, worst properties should be assumed under the heavily loaded regions and the best properties under the lightly loaded regions.

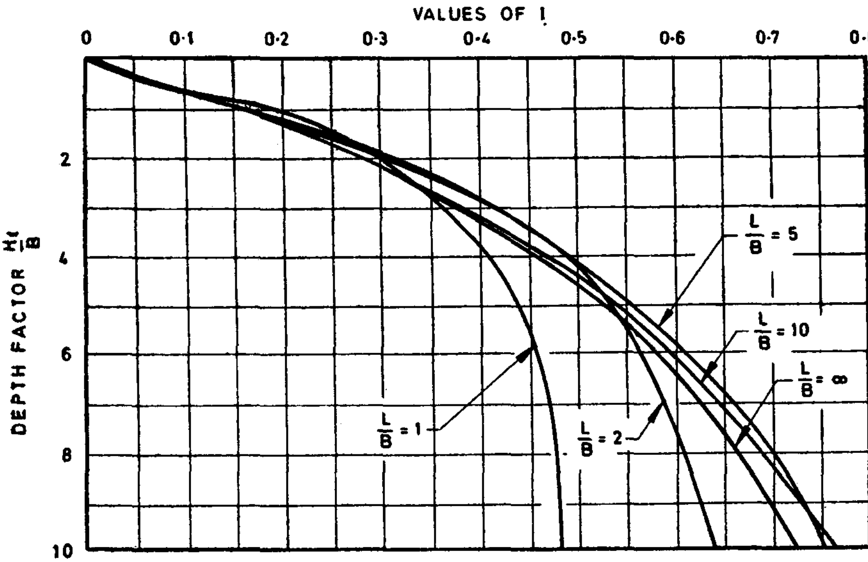


FIG. 11 STEINBRENNER'S INFLUENCE FACTORS FOR SETTLEMENT OF THE CORNER OF LOADED AREA $L \times B$ ON COMPRESSIBLE STRATUM OF $\mu = 0{\cdot}5$, THICKNESS H_t

9.4.2 If the variation occurs over distances lesser than half the width of foundation, the settlement analysis should be based on worst and average conditions. That is, the worst properties should be assumed under the heavily loaded region and the average properties under the lightly loaded regions.

9.5 Correction for Depth and Rigidity of Foundation on Total Settlement

9.5.1 *Effect of Depth of Foundation* — The relevant equation in **9.1** and **9.2** are applicable for computing the settlement of foundations located at surface. For the computation of settlement of foundations founded at certain depth, a correction should be applied to the calculated S_f in the form of a depth factor to be read from Fig. 12.

Corrected settlement $= S_{fd} = S_f \times$ Depth factor .. .. (12)

9.5.2 *Effect of the Rigidity of Foundation* — In the case of rigid foundations, for example, a heavy beam and slab raft or a massive pier, the total settlement at the centre should be reduced by a rigidity factor.

$$\text{Rigidity factor} = \frac{\text{Total settlement of rigid foundation}}{\text{Total settlement at the centre of flexible foundation}}$$

$$= 0{\cdot}8 \quad \ldots \quad (13)$$

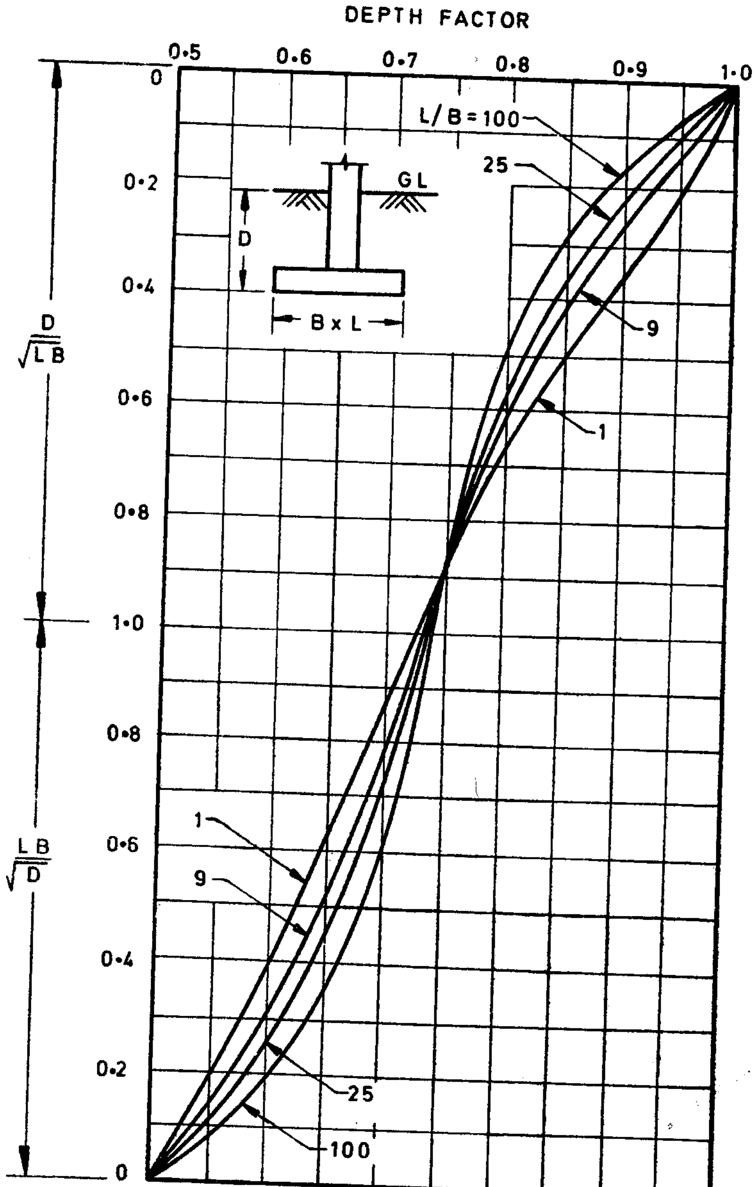


FIG. 12 FOX'S CORRECTION CURVES FOR SETTLEMENTS OF FLEXIBLE RECTANGULAR FOOTINGS OF $L \times B$ AT DEPTH D

9.6 Differential Settlement — It is usually the differential settlement rather than the total settlement that is required for designing of a foundation. But it is more difficult to estimate the differential settlement than the maximum settlement. This is because, the magnitude of the differential settlement is affected greatly by the non-homogeneity of natural deposits and also the ability of the structures to bridge over soft spots in the foundation. On a very important job, a detailed study should be made of the sub-soil profile and the relation between foundation movement and forces in the structure should be investigated as indicated in **4.1** and **9.4**. Ordinarily, it is sufficient to state the design criteria in terms of allowable total settlements and design accordingly.

10. ESTIMATION OF TIME RATE OF SETTLEMENT

10.1 The settlement at any time, may be estimated by the application of the principles of Terzaghi's one dimensional consolidation theory. Based on this theory, the total settlement at time t, is given by:

$$S_t = S_i + US_c \quad \ldots \quad \ldots \quad \ldots \quad (14)$$

where

U = degree of consolidation

$$= F(T); \quad \ldots \quad \ldots \quad \ldots \quad (15)$$

$$T = \frac{c_v\ t}{H^2}; \quad \ldots \quad \ldots \quad \ldots \quad (16)$$

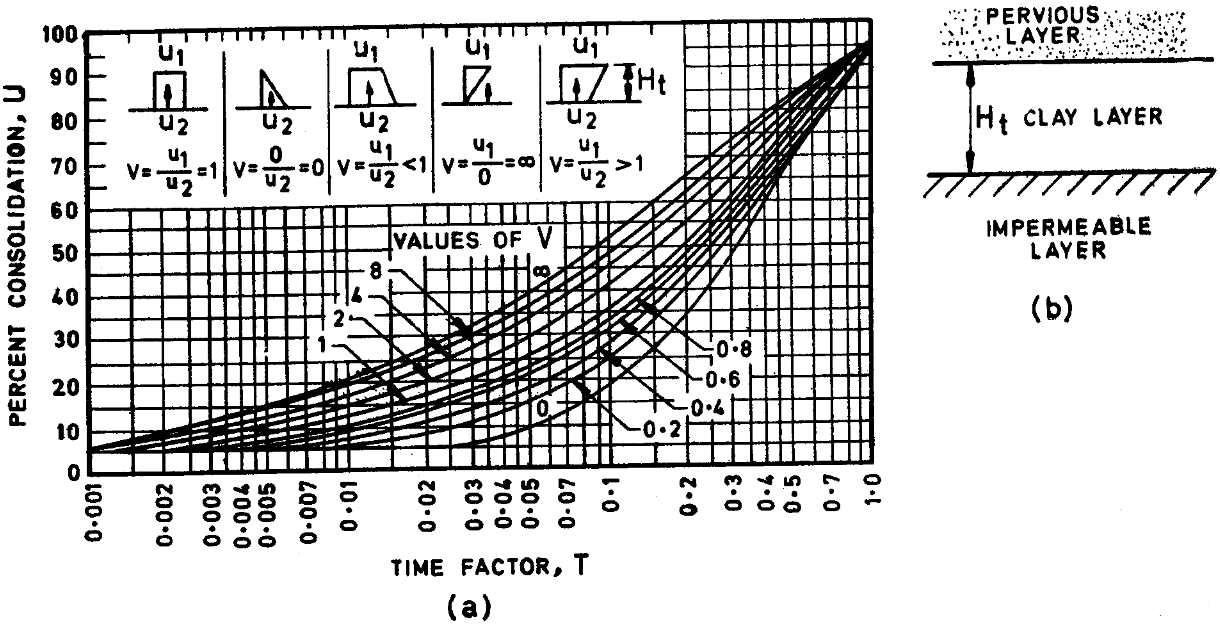
t = time at which the settlement is required, years; and

c_v = average coefficient of consolidation over the range of pressure involved obtained from an oedometer test, m^2/year.

The relationship between T and U in equation (16), depends on pressure distribution and nature of drainage. This relationship is shown in Fig. 13. When considering the drainage of clay layer, concrete of foundation may be assumed as permeable. The coefficient of consolidation shall be evaluated from the one dimensional consolidation tests using suitable fitting methods [*see* IS : 2720 (Part XV)-1965*].

10.2 In the case of evaluation of time rate of settlement of structures constructed with certain construction time, the procedure illustrated in Appendix D may be followed.

*Methods of test for soils: Part XV Determination of consolidation properties.



NOTE — In the case of open layer with two way drainage for all valves of $\frac{u_1}{u_2}$ use the curve for $\frac{u_1}{u_2} = 1$.

FIG. 13 RELATIONSHIP BETWEEN PERCENT CONSOLIDATION AND TIME FACTOR

APPENDIX A

(*Clauses* 8.1.2 *and* 8.2.2)

DETERMINATION OF PRE-LOADING PRESSURE CONDITIONS AND IMPLICATIONS

A-1. GEOLOGICAL INFORMATION AND PORE PRESSURE

A-1.1 A geological investigation of a site may yield information on the age of various strata, the time which has elapsed since there has been deposition of soil at site, the erosion that has occurred at the site in the past ages, the water table and artesian conditions and other matters which may aid in determining which of the cases outlined in **8.1** holds in any given case.

A-1.2 Such sources of information are largely qualitative and often not sufficient for consolidation analysis. There are several methods for the quantitative estimation of the maximum intergranular pressure that the soil has ever undergone. In **A-2**, a method based on consolidation test data is detailed.

A-2. DETERMINATION OF MAXIMUM INTERGRANULAR PRESSURE

A-2.1 In this method, point '*O*' of maximum curvature is located on the e-log $\bar{p}$ curve (*see* Fig. 14). Through this point, three lines are drawn; first the horizontal line *OC*, then the tangent *OB* and finally the bisector *OD* of the angle formed by the first and second lines. The straight line portion of the e-log $\bar{p}$ curve is produced backwards and its intersection with *OD* gives point *E*. The $\bar{p}$ value ($\bar{p}_c$) corresponding to point *E*, is the maximum intergranular pressure.

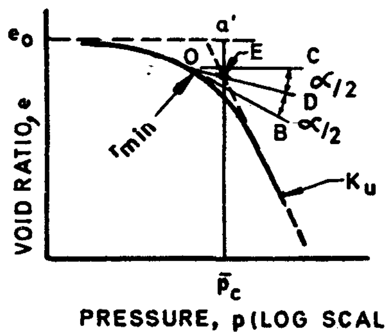


FIG. 14 GRAPHICAL CONSTRUCTION FOR DETERMINING FORMER MAXIMUM PRESSURE $\bar{p}_c$ FORM e-log p CURVE

A-2.2 For each sample which has been tested in the laboratory, the maximum previous intergranular pressure may be determined by this method. These values may then be plotted at appropriate depths on the effective pressure diagrams shown in Fig. 7.

A-2.3 The past pressure curve may then be compared with the effective pressure curve for simple static case (line *db*). If the curves coincide, a simple static case of a normally consolidated deposit is indicated. If there is agreement at top and at the bottom of the clay stratum (line *deb*), but the past pressure curve falls to the left at the centre of the stratum, residual hydrostatic excess is indicated. If there is agreement at top but the past pressure curve falls to the left at the bottom (line *df*), one of the following conditions is indicated:

a) The stratum may have drainage at the top surface only, the case being one involving residual hydrostatic excess; and

b) There may be double drainage and an artesian condition. The boring data should be clearly analysed to decide which of the two above cases holds. If the past pressure curve (line *gh*) falls to the right of static intergranular curve then a case of precompression exists.

A-2.4 Limitation of the Methods — In principle, the procedure for comparison explained above is simple. In actual practice, if the sample has been disturbed to an appreciable degree during or after sampling, the pressures obtained by the method laid down in **A-2**, tend to be too low. If the samples are undisturbed, the results obtained are satisfactory.

A-3. DETERMINATION OF INITIAL EFFECTIVE STRESS AND PORE PRESSURE

A-3.1 General — The total vertical pressure at any depth shall be computed from the unit weights of the overlying material. In the case of pervious layers, the pore pressures and consequently the effective stresses can be estimated from ground water level observation. In the case of clay layers the type of preloading conditions is identified by the geological investigations and by the estimation of the maximum previous intergranular pressure; and the pore pressures and effective stresses may be estimated as given in **A-3.2** to **A-3.6**.

A-3.2 Simple Static Case — In the simple static case, the pore pressures are equal to the hydrostatic pressures. The effective stress is computed as the difference between the total and neutral pressure. The effective stress may also be determined directly using the submerged unit weights below water table.

A-3.3 Residual Hydrostatic Case — In the residual hydrostatic case, the present effective stress will be equal to the maximum previous intergranular pressure, unless there was scope in the geologic history for the soil layer to have undergone a greater load than the present overburden. The pore pressure will be equal to the total stress minus the effective stress. The portion of the pore pressure in excess of the hydrostatic pressure, will ultimately be transferred to the soil grains and, therefore, in the pressure increment, the excess pore pressure should be added to the pressure increment due to the imposed loads.

A-3.4 Artesian Case — In the artesian case, the excess hydrostatic pressure may be assumed to vary linearly within the clay layer. Then, the effective stress will be equal to the total pressure minus the sum of the hydrostatic and excess hydrostatic pressure. The maximum intergranular pressure estimated in **A-2** will be equal to this pressure unless there was scope for precompression in the geological history of the site. The pressure increment in this case will be equal to the pressure increment due to building load alone unless, the artesian pressure is reduced either by man made or natural causes, in which case, the change in stress due to change in artesian pressure should be added to the pressure increment due to building loads.

A-3.5 Precompressed Case — In the precompressed case, the effective stress and pore pressure will be the same as in the simple static case.

A-3.6 Complex Cases — There are situations where the cases can be the combinations of the four simple cases indicated in **A-3.2** to **A-3.5**. Such complex cases require individual treatment and are not dealt with in this standard.

APPENDIX B

(*Clause* 8.3.6)

SELECTION OF STRESS DISTRIBUTION THEORIES TO COMPUTE THE PRESSURE INCREMENT

B-1. GENERAL

B-1.1 The methods generally used for determination of the pressures induced by building loads are based on the mathematical model due to Boussinesq who assumed isotropy, homogeneity and elastic half space conditions.

B-1.2 With the assumptions, given in **B-1.1** equation for the vertical stress σ_z, at a point N (*see* Fig. 15) due to the application of concentrated load p, at the surface of the soil is:

$$\sigma_z = \frac{3p}{2\,\pi z^2} \cos^5\beta \quad \ldots \quad \ldots \quad \ldots \quad \text{(B 1)}$$

where

z = vertical co-ordinate, and
β = angular co-ordinate.

B-1.3 With the assumptions given in **B-1.1**, the computation of vertical normal stress σ_z due to a uniformly distributed load p on a circular area with radius R at a depth z below the centre (*see* Fig. 16) of the loaded area, may be obtained from:

$$\sigma_z = p\left[1 - \frac{1}{1+(R/z)^2}\right]^{3/2} \quad \ldots \quad \ldots \quad \ldots \quad \text{(B 2)}$$

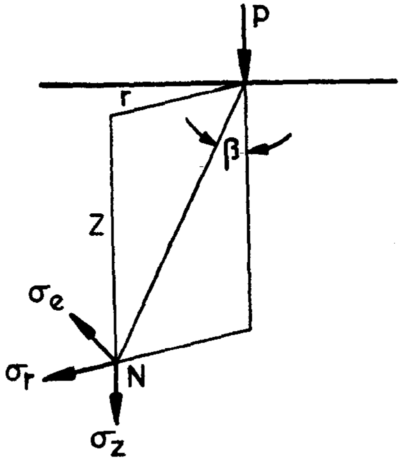


FIG. 15 STRESSES AT POINT *N*

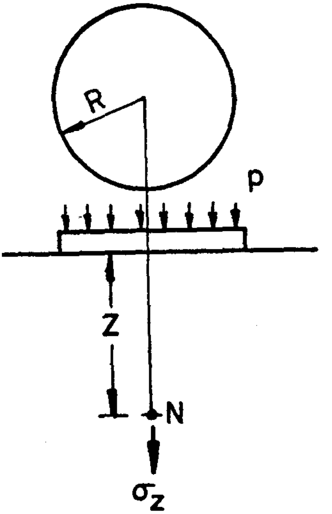


FIG. 16 VERTICAL STRESS σ_z BELOW THE CENTRE OF CIRCULAR FOOTING

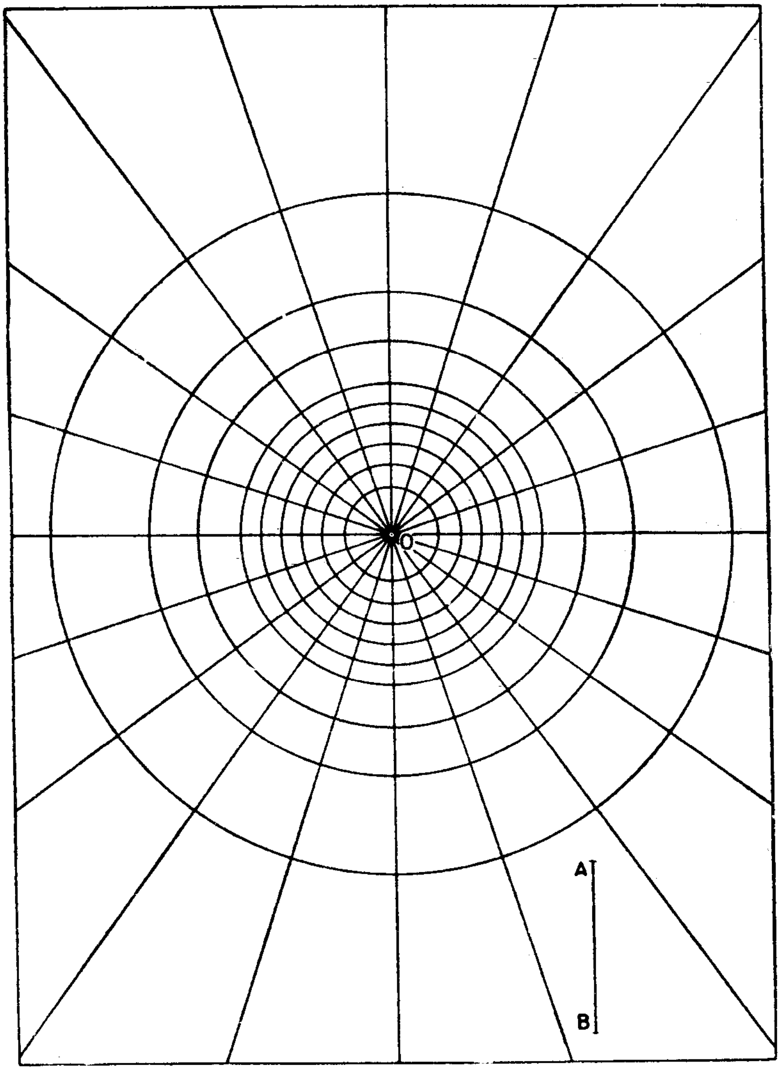
This equation has been represented in a chart form by Newmark and is shown in Fig. 17. This chart may be used to estimate the vertical normal stress at a depth z below a point N, due to a uniformly distributed load q on any area of known geometry. The procedure is to draw the given loaded area on the chart with the point below which the stress is required coinciding with the centre O of the chart. The scale should be chosen so that the depth z at which the stress is required is equal to unit distance AB marked in the chart. In the chart, one influence area is defined as the area included between two consecutive radial lines and circular area. The number of influence areas enclosed in the chart by the given loaded area are counted and the stress σ_z is then estimated as:

$$\sigma_z = p \times I_B \times \text{number of influence areas} \quad \ldots \quad \ldots \quad \text{(B 3)}$$

where

p = intensity of given loading, and

I_B = influence value marked at the bottom of the chart (Boussinesq, Newmark).



Influence value = 0·005

FIG. 17 INFLUENCE CHART FOR VERTICAL PRESSURE (BOUSSINESQ CASE)

B-1.3.1 Alternatively, the vertical normal stress σ_z at a point N at depth z below the corner of a rectangular loaded area with a uniformly distributed load, may also be estimated using the chart shown in Fig. 18. From the chart σ_z is estimated from the expression:

$$\sigma_z = p \times I_B \quad \text{.. .. .. ..} \quad (B\ 4)$$

where

I_B = a function of L/z and B/z,

L = length of the loaded area, and

B = width of the loaded area.

B-2. NORMALLY CONSOLIDATED CLAYS

B-2.1 From the point of view of compressibility, normally consolidated clays may be considered as effectively isotropic and, therefore, the Boussinesq solution given in **B-1** is applicable.

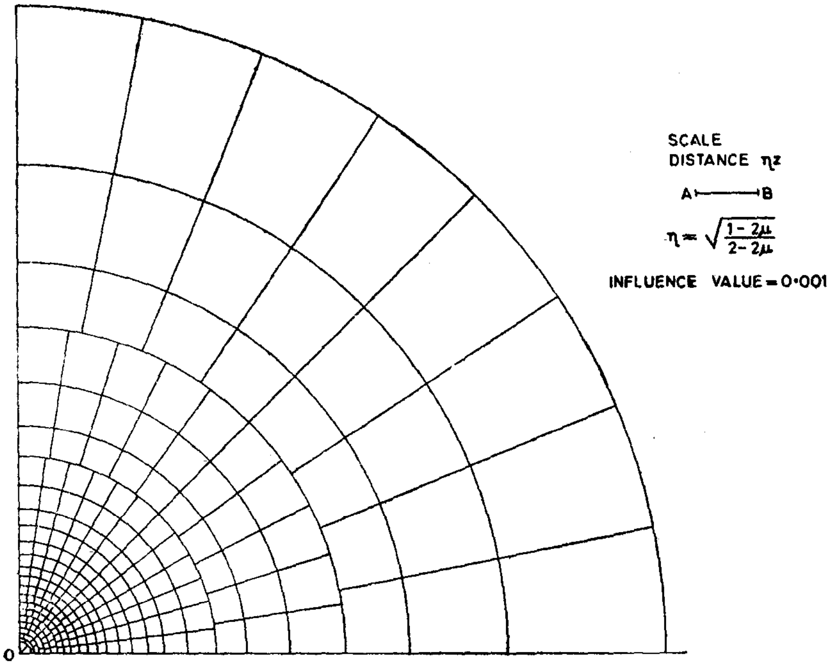


FIG. 19 INFLUENCE CHART FOR UNIFORM VERTICAL NORMAL STRESS (WESTERGAARD SCALE)

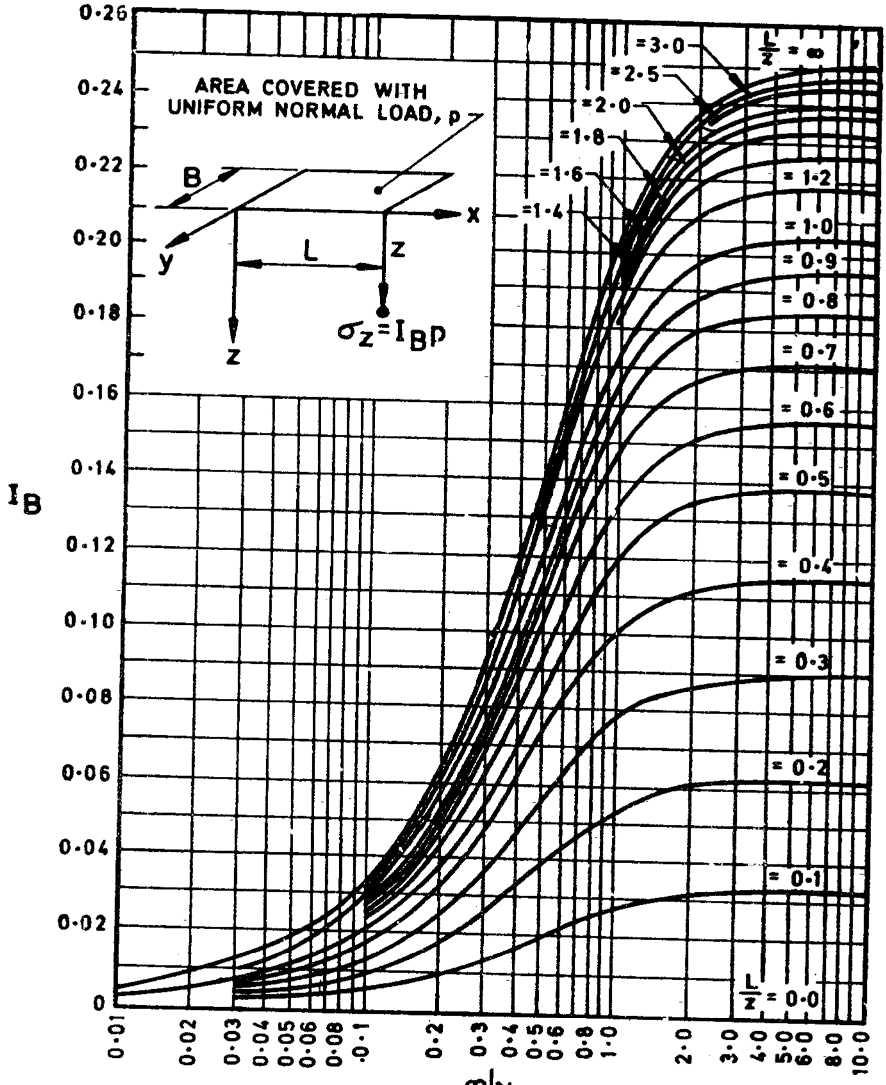


FIG. 18 CHART FOR RECTANGULAR AREA UNIFORMLY LOADED (BOUSSINESQ CASE)

B-3. PRECOMPRESSED CLAYS

B-3.1 Many overconsolidated and laminated clays can be expected to exhibit marked anisotropy, particularly if the laminations are varved, and this condition satisfies the assumption of Westergaard that $\frac{E_h}{E_v}$ = infinity, where E_h and E_v are the Young's moduli in the horizontal and vertical directions.

B-3.2 The influence chart, to calculate the normal stresses σ_z at a point, with a depth z below the ground surface using Westergaard's solution is shown in Fig. 19. The vertical stresses σ_z is calculated using a procedure similar to that illustrated for Fig. 17. The unit distance AB marked in the chart corresponds to a depth ηz.

where $\eta z = \sqrt{\frac{1-2\mu}{2-2\mu}}$ and μ is the Poisson's ratio for the soil.

B-4. SAND DEPOSITS

B-4.1 In the case of sands, experimental evidence indicates that $\frac{E_h}{E_v}$ is considerably less than 1. Hence, sands present the case of a model, which is perfectly elastic and isotropic in every horizontal direction. Published experimental evidence suggests that the distribution of σ_z in sand can be reasonably computed from the semi-empirical equation of Fröhlich with a concentration factor or index $m' = 4$.

B-4.2 For estimating the normal stress σ_z at a depth z below the surface of the soil with decreasing compressibility with depth, due to a uniformly distributed load on a given area, the chart shown in Fig. 20 may be used. The procedure for using the chart is the same as illustrated for Fig. 17.

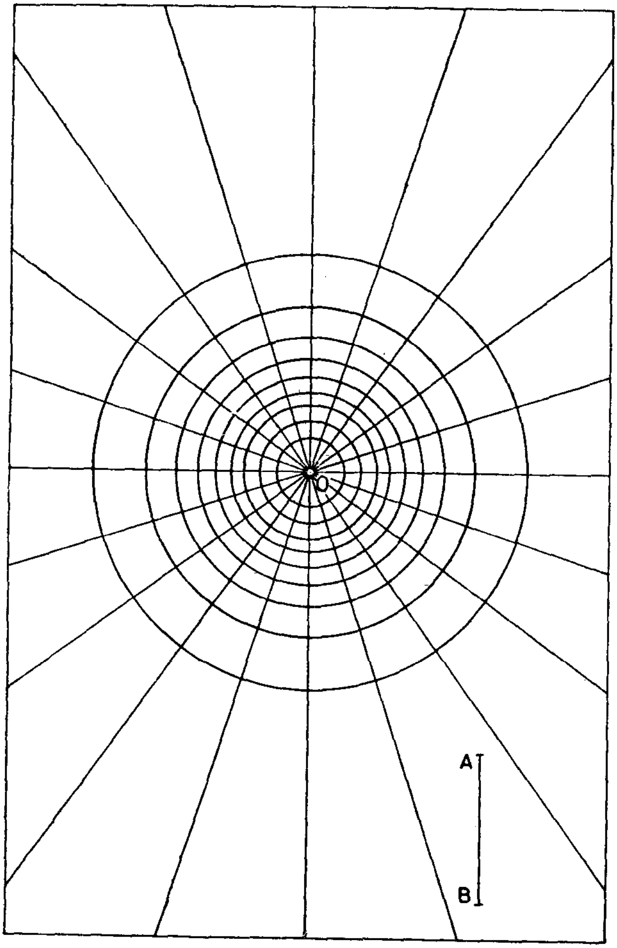
B-5. VARIABLE DEPOSITS

B-5.1 In the present state of knowledge, variable deposits are to be treated as a simple, homogeneous, isotropic and elastic case for purposes of computation of vertical stresses.

B-6. LIMITATIONS

B-6.1 In many instances, it is difficult to make definite statements regarding the accuracies obtainable when formulae based on the theory of elasticity are used for stress determinations in soil masses. The solutions from elastic theory which have been mentioned earlier are rigorously correct only for materials in which stresses and strains are proportional. Moreover each of the above solutions is valid only for the specific conditions upon which it is based. When the above solutions are used for estimating stresses in soils, the inaccuracies that occur because the soils are not elastic, are of unknown magnitude and are not well understood. Till such time a clear picture of a satisfactory generalised stress-strain relationship in soils is evolved, the following broad guidelines may be followed in estimating the induced stresses in soil masses due to applied surface loads:

a) Normally and lightly overconsolidated clays : E_h/E_v approximately one — Boussinesq solution



Influence value = 0·005

FIG. 20 INFLUENCE CHART FOR VERTICAL STRESS (FRÖHLICH CONCENTRATION FACTOR m′=4)

b) Heavily overconsolidated clays $1{\cdot}5 < E_h/E_v < 3$	Westergaard
c) Sands $E_h/E_v < 1$	Fröhlich with $m'=4$
d) Variable deposits	Boussinesq solution

APPENDIX C

(*Clauses* 9.2.2.2 *and* 9.2.2.3)

PROCEDURE FOR OBTAINING THE FIELD COMPRESSION CURVE

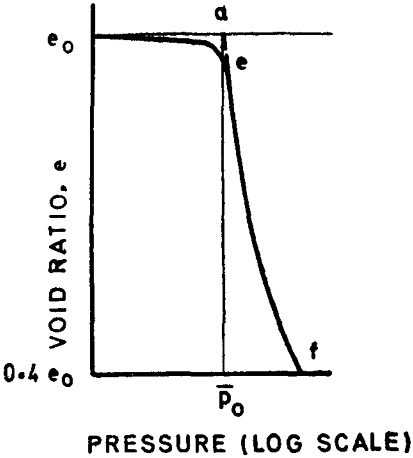
C-1. PROCEDURE FOR NORMALLY CONSOLIDATED CASE

C-1.1 In the normally consolidated case, if the soil is extra sensitive, the projection of the bottom portion of the laboratory compression curve will meet the e_0 line at the point a (e_0, $\bar{p}_0$) or very close to it, as shown in (a) in Fig. 21. Therefore, it may be taken that the line aef represents the field compression curve. If the clay is of ordinary sensitivity, the field curve is given by af where a is the point (e_0, $\bar{p}_0$) and f is the point where the laboratory compression curve meets the $e=0{\cdot}4e_0$ line as shown in Fig. 21(b).

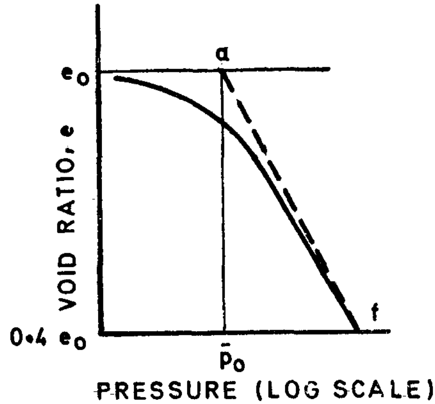
C-2. PROCEDURE FOR OVERCONSOLIDATED CASE

C-2.1 The procedure for obtaining the field compression curve from the laboratory oedometer curve is as follows.

C-2.1.1 The construction requires unloading the sample in increments after the maximum pressure has been reached, in order to obtain a laboratory rebound curve. The laboratory curve is represented by K_u in Fig. 21(c). Point b represents the void ratio e_0 and effective overburden pressure $\bar{p}_0$ of the clay as it existed in the ground before sampling. The field e-log p curve should pass through this point. The vertical line $\bar{p}_c$ corresponds to the maximum consolidation pressure as determined by the graphical construction given in **A-2**. The portion of the field e-log p curve between $\bar{p}_0$ and $\bar{p}_c$ is a recompression curve. Since in the laboratory there is little difference in slope between rebound and recompression curves, it is assumed that the field curve between $\bar{p}_0$ and $\bar{p}_c$ is parallel to the laboratory rebound curve. Accordingly, a line is drawn from b parallel to cd; its intersection with the vertical at $\bar{p}_c$ is denoted by a'. The field curve for pressures above $\bar{p}_c$ is approximated by the straight line $a'f$, where f is the intersection of the downward extension of the steep straight portion of K_u and the horizontal line $e=0{\cdot}4\,e_0$. Between b and a' a smooth curve is sketched in as indicated in Fig. 21(c).

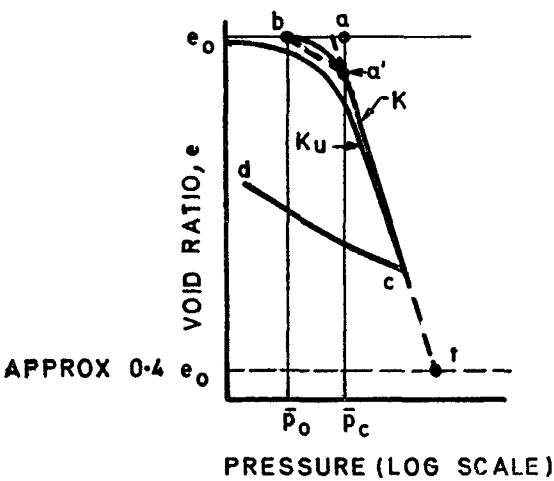


21A Normally consolidated case



21B Clay of ordinary sensitivity

IB



21C Overconsolidated case

FIG. 21 GRAPHICAL CONSTRUCTION FOR APPROXIMATING FIELD RELATION BETWEEN e AND p

APPENDIX D

(*Clause* 10.2)

TIME RATE OF SETTLEMENT DUE TO CONSTRUCTION TYPE OF LOADING

D-1. GENERAL

D-1.1 In the construction of a typical structure, the application of load requires considerable time, and the loading progress may be shown graphically by a diagram, such as the upper curve of Fig. 22. The net load does not become positive until the building weight exceeds the weight of excavated material. The time at which this occurs is represented by point *A* of the figure, and it will be assumed that no appreciable compression of the underlying strata will occur until this point is reached. The time from this point until the building is completed will be designated as the loading period. It may be assumed that the excavation and the replacing of an equivalent load have no effect on settlement. Actually some rebound and some recompression always occur during this period, and they can be studied in detail, but often they are not considered to be of sufficient importance.

D-2. APPROXIMATE METHOD DUE TO TERZAGHI

D-2.1 The time settlement curve for the given case, on the basis of instantaneous loading, is obtained by the consolidation theory and shown (line *OCD* of Fig. 22), zero time being measured from point *A*. In the loading interval, the loading diagram may be approximated by a straight line as shown by the dotted line in Fig. 22. According to the assumption made in this method, the settlement at time t_1 is equal to that at time $1/2\ t_1$ on the instantaneous loading curve. Thus, from point *C* on the instantaneous curve, point *E* is obtained on the corrected curve.

D-2.2 At any smaller time t the settlement determination is as follows.

D-2.2.1 The instantaneous curve at time $t/2$ shows the settlement *KF*. At time t the load acting is t/t_1 times the total load, and the settlement *KF* should be multiplied by this ratio. This may be done graphically as indicated in the figure. The diagonal *OF* intersects time t and point *H*, giving a settlement at *H* which equals $(t/t_1) \times KF$. Thus *H* is a point on the settlement curve, and as many points as desired may be obtained by similar procedure.

D-2.3 Beyond point *E* the curve is assumed to be the instantaneous curve *CD*, offset to the right by one half of the loading period; for example, *DJ* equals *CE*. Thus, after construction is completed, the elapsed time from

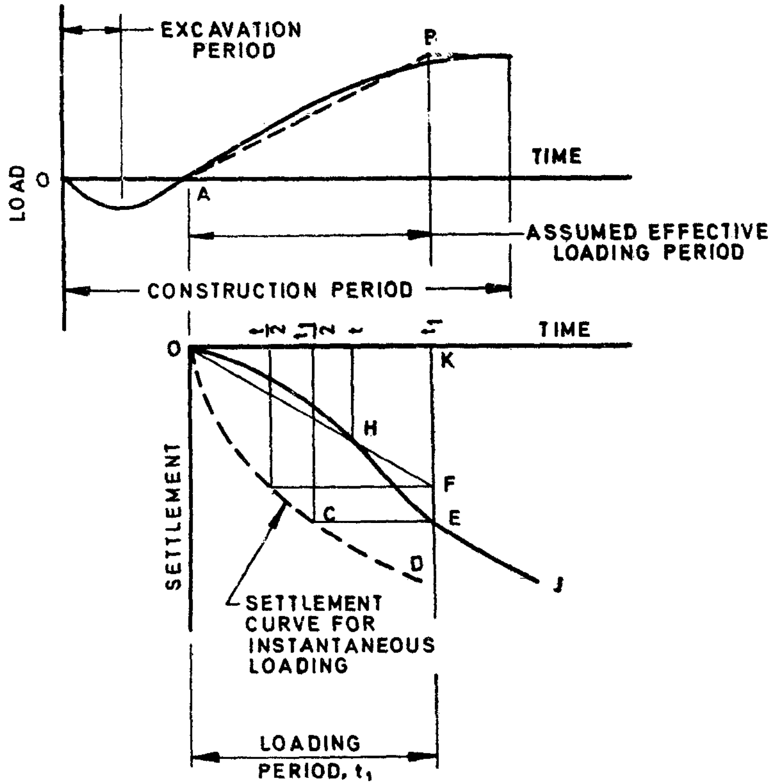


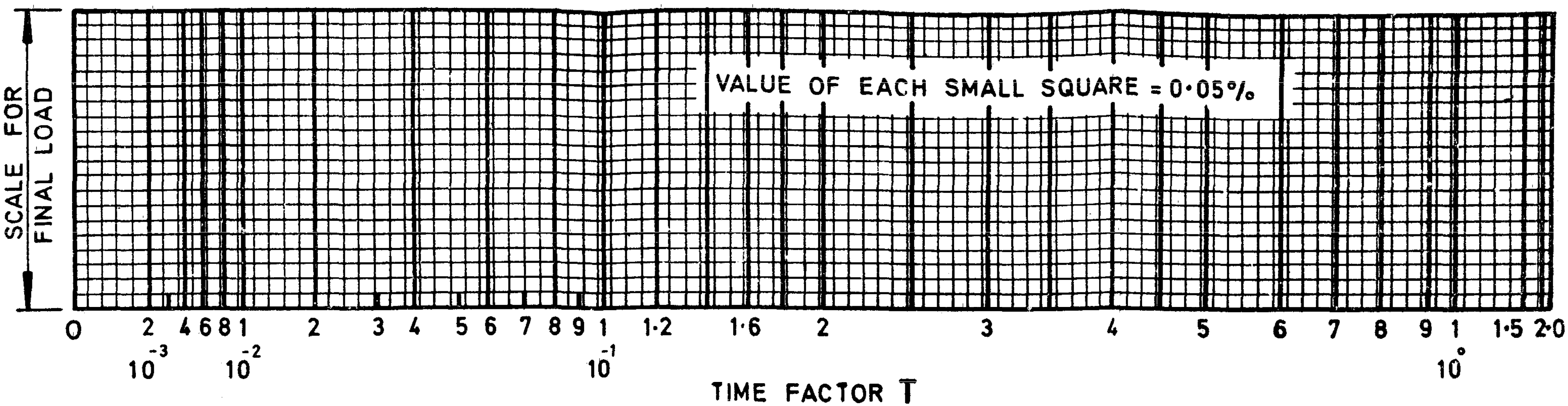
FIG. 22 TERZAGHI'S APPROXIMATE METHOD

the start of loading until any given settlement is reached is greater than it would be under instantaneous loading by one half of the loading period.

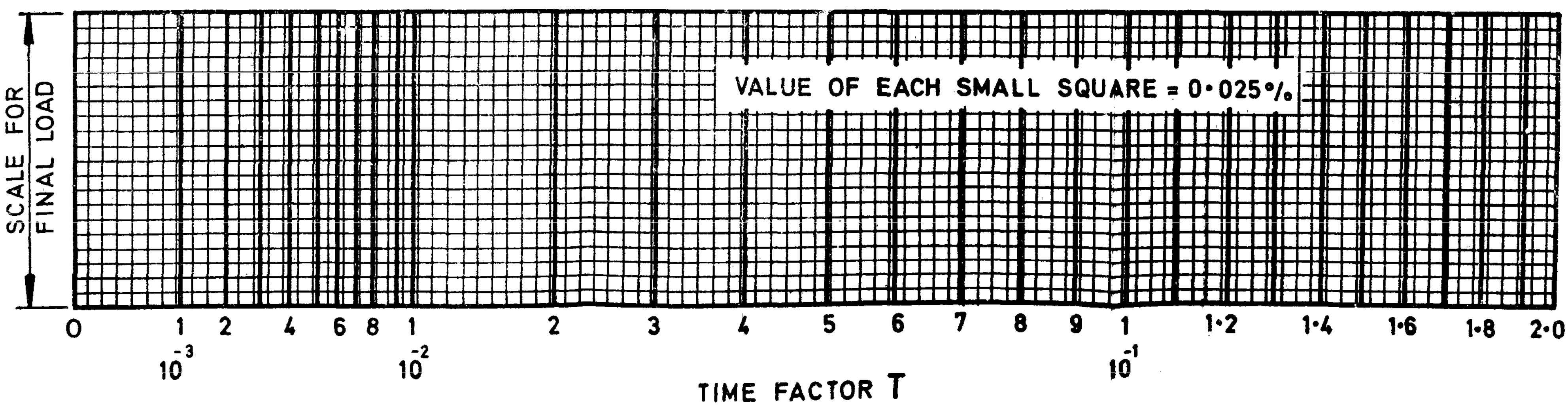
D-3. GRAPHICAL SOLUTION

D-3.1 An alternative graphical solution for construction type of loading, does not involve an approximation of the loading diagram with linear variation as in the previous method. In this method, the charts shown in Fig. 23A and 23B are used for this purpose. The small squares in the chart represent a given fraction of consolidation as noted in the particular chart. The value of each small square in Fig. 23B is only half of the corresponding value of Fig. 23A and hence the former may be used if the consolidation is less than 50 percent so that the accuracy of the final result can be increased. The horizontal axis represents the time factor. The vertical axis is the load axis. Let the time load curve be as shown in Fig. 24. The time axis is first replaced by the time factor axis. Then, the consolidation at time t_1 is determined as follows.

D-3.2 The load time factor curve is drawn on the chart as a mirror reflection starting from the time factor T_1 corresponding to time t_1. It is to be noted that the mirror reflection is to be obtained by warping the time factor axis of the time load curve to suit the corresponding scale in the reflected part of the chart, as shown in Fig. 24. The number of squares under this curve multiplied by the value of each square is equal to the consolidation at time t_1.



23A Chart with value of each small square = 0·05%



23B Chart with value of each small square = 0·025%

FIG. 23 CHART FOR FINDING CONSOLIDATION DUE TO CONSTRUCTION TYPE LOADING

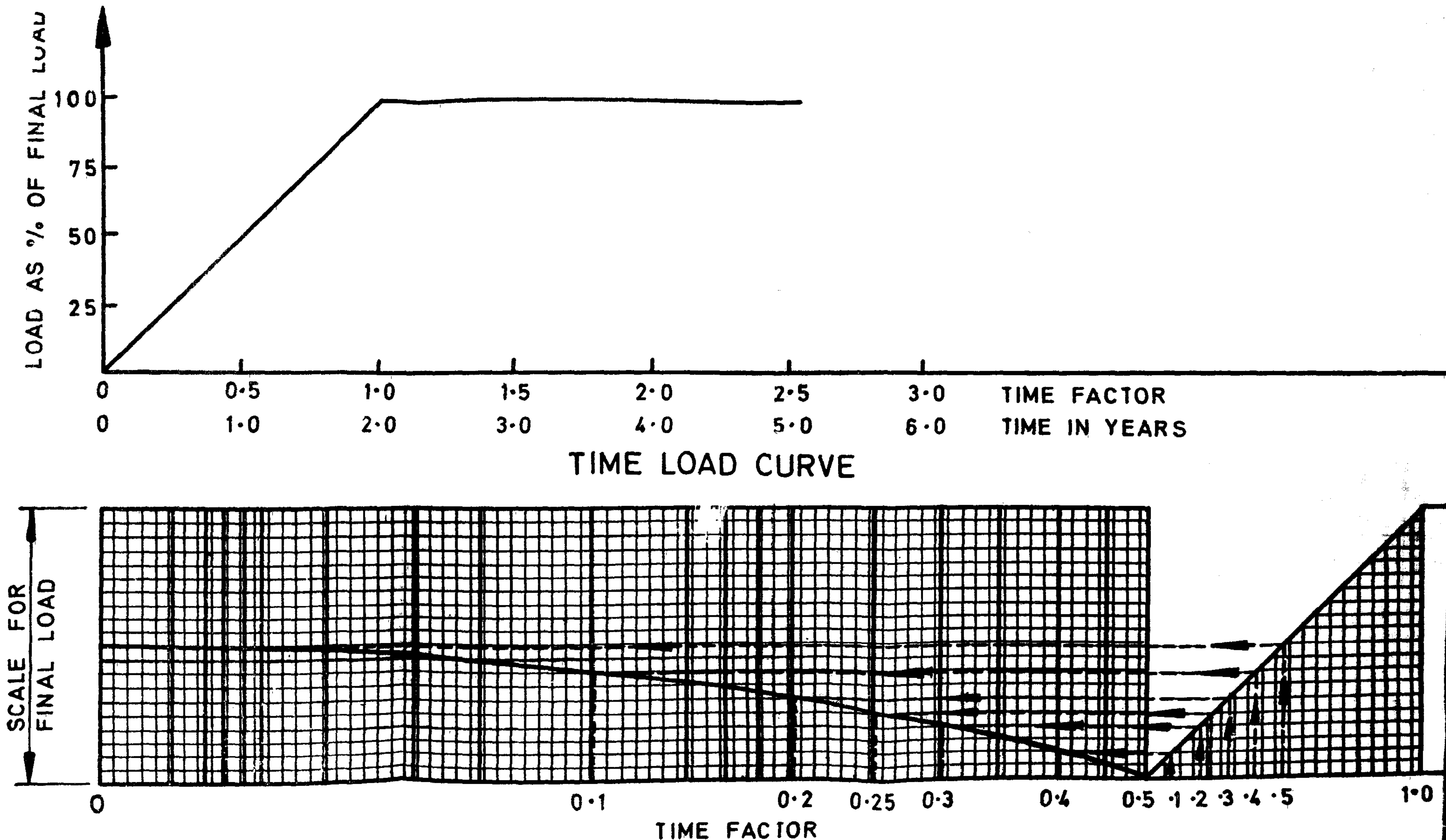


FIG. 24 USE OF CONSOLATION CHART

AMENDMENT NO. 1 MAY 1981

TO

IS : 8009 (Part I)-1976 CODE OF PRACTICE FOR CALCULATION OF SETTLEMENTS OF FOUNDATIONS

PART I SHALLOW FOUNDATIONS SUBJECTED TO SYMMETRICAL STATIC VERTICAL LOADS

Corrigenda

(*Page* 6, *clause* **3.0**, *line* 5) — Substitute 'C' for 'c'.

(*Page* 15, *clause* **9.1.2**, *equation* 2) — Substitute 'C' for 'c'.

(*Page* 22, *Fig*. 12, *legend*) — Substitute '$\frac{\sqrt{LB}}{D}$' *for* '$\frac{LB}{\sqrt{D}}$'

(*Page* 32, *clause* **B-3.2**, *last line*) — Substitute 'η' *for* 'ηz'.

Alterations

(*Page* 7, *clause* **3.0**, *symbol* S_1) — Delete along with its explanation.

(*Page* 12, *clause* **7.1**, *line* 2) — Substitute 'IS : 1892-1979*' *for* 'IS : 1892-1962*'.

(*Page* 12, *foot-note with* '*' *mark*) — Substitute the following for the existing foot-note:

'*Code of practice for sub-surface investigations for foundation (*first revision*).'

(*Page* 15, *clause* **9.1.2**, *lines* 2 *and* 3) — Substitute 'IS : 4968 (Part III)-1977*' *for* 'IS : 4968 (Part III)-1971*'.

(*Page* 15, *foot-note with* '*' *mark*) — Substitute the following for the existing foot-note:

'*Method for subsurface sounding for soils : Part III Static cones penetration test (*first revision*).'

(*Page* 16, *clause* **9.1.3**) — Substitute the following for the existing clause:

'**9.1.3** *Method Based on Plate Load Test* — Plate load test should be performed at the proposed foundation level and the settlement of a square plate under the design intensity of loading on the foundation estimated (*see* IS : 1888-1971*). Then the total settlement of the proposed foundation is given by:

$$S_f = S_p \left[\frac{B\ (B_p + 30)}{B_p\ (B + 30)} \right]^2$$

NOTE — If the water table is at shallow depths or if it is expected that the water table for the foundations will be different from what is obtained at the time of the field load test, then, the effect of water table on the bearing capacity of the foundations may be different from that for the load test. Therefore, the water table correction given in Fig. 9 may be suitably adopted for the load test and the foundations '.

(*Page* 16, *clause* **9.1.4**, *last sentence*) — Substitute the following for the existing sentence:

' If the water table is at a shallow depth, the settlement read from Fig. 9 shall be divided, by the correction factor W' read from the inset in the same figure. '

(*Page* 16, *clause* **9.2.2.1**) — Substitute the following for the existing clause:

' **9.2.2.1** For situations shown in Fig. 2 it may be assumed that:

$$S_i = 0, \text{ and}$$
$$S_c = S_{oed}$$

NOTE — To be exact, the settlement of the sand layers may also have to be added to the computed value of S_f . The designer has to use his discretion whether to add the settlement due to the sand layers or not.

The details of the computations of S_{oed} depends upon the preloading conditions. The details are given in **9.2.2.2** to **9.2.2.4**.'

Addendum

(*Page* 7, *clause* **3.0**) — Add the following symbols at appropriate places:

' S_p = Settlement of test plate under a given load intensity per unit area, cm

B_p = Size of test plate, cm '

(BDC 43)

Printed at Simco Printing Press, Delhi, India

AMENDMENT NO. 2 JULY 1990

TO

IS 8009 (Part 1) : 1976 CODE OF PRACTICE FOR CALCULATION OF SETTLEMENTS OF FOUNDATIONS

PART 1 SHALLOW FOUNDATIONS SUBJECTED TO SYMMETRICAL STATIC VERTICAL LOADS

[*Page* 28, *clause* **B-1.3**, *expression* (B 2)] — Substitute the following for the existing matter:

$$\sigma_z = p\left[1 - \left\{ \frac{1}{1 + (R/z)^2} \right\}^{3/2} \right]$$

(BDC 43)

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