# Computer Solution for Settlement of Foundations

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Modern complex structures with heavy loadings require a detailed analysis of the differential settlements to be expected because of soil consolidation. However, the accurate calculation of such settlements considering variable soil properties and many loaded points is a laborious and time-consuming job. It was decided to develop a digital computer solution which would be based on the one-dimensional consolidation theory and the elastic theory of stress distribution but would still take into account the variation in soil properties and the complexity of many loading conditions.

The mathematical integration of the expression developed to predict settlement is very difficult because the stress-strain relation in soil is nonlinear, there is no general relation between the pressure at a point in soil and the many loads that cause the pressure, and both the initial and final pressures are variables. To overcome these difficulties, a numerical modified Euler method was used. A flow diagram and a specific FORTRAN language instruction for the IBM 1620 computer were prepared and typical differential settlement problems were solved. The settlements obtained compared favorably with experimental results. The time involved to execute a program depends on the computer speed, the number of loads, thickness of soil layer and the increment of thickness selected. The computer solution compares favorably in cost, accuracy and speed with other calculation methods. The greater the complexity of the structure, the better is the comparison.

•A STRUCTURE usually rests on soil and the load which is transmitted by the foundations will cause the soil to undergo compressive strains resulting in the settlement of the structure. Therefore, the stress-strain characteristics of the foundation soil must be studied to understand the behavior of the structure and to predict with a fair degree of accuracy the probable settlement during the life of the structure, so that suitable provisions may be made to take care of the settlement.

Settlement of itself does not affect a structure adversely. In fact, all structures, with the exception of those whose foundation reactions are transmitted to solid rock, will settle to a greater or lesser extent. As long as settlement is uniform throughout the loading plan of a structure and it does not reach excessive proportions, one does not have to fear the possible failure or malfunction of the structure. But if the settlement is uneven, i.e., if one corner of a building or one end of a structure settles more than the others, serious consequences may ensue. The progressive buildup of unequal settlements may eventually result in overturning or leaning of the structure. In some cases, the integrity of the framework may be destroyed. Unequal settlement may also cause serious overstress in some members and subject the structure to a loading pattern not provided for in the original design, thus making the structure dangerously unstable or unsafe.

From the foregoing discussion, it is evident that the most important effect of settlement is not the total magnitude of settlement which the structure may undergo but the

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differential settlement of various portions of the structure and the resulting distribution of pressures on the soil and of the foundation reactions on the structure. It is, therefore, essential to develop a method to analyze the distribution of settlement of the loading plan of a structure. Once this distribution of settlement is known, suitable measures can be taken to guard against serious consequences. Alternatively, the loading plan may have to be radically changed.

For a comparatively simple structure, the evaluation of the distribution of settlements does not present any difficulties. But complex structures of the types gaining in popularity in the modern age require a lengthy computational procedure which would almost invariably involve computation errors. When dealing with complex structures, one cannot afford the luxury of a mistake. Therefore, it is both expedient and necessary to carry out the computations with an electronic computer. The present study develops a method of analysis which enables the computation of the distribution of settlements through a digital computer program.

## SCOPE

Settlement is affected by the pressure distribution within the soil mass. The solution presented here is based on an original idea by Stoll (4) and on the consolidation theory developed by Terzaghi to explain the compression of soil under structural loads. Several consolidation theories have been published but to include all theories along with complex soil properties would make the problem unnecessarily long. However, for the one consolidation theory, the study does consider the process of evaluating settlements in soils with variable properties.

## THEORETICAL ANALYSIS OF SETTLEMENT

Settlement of a structure resting on soil may be caused by two main types of action within the foundation soil. The first is a shearing failure within the soil mass. This causes the soil mass to slide downwards and laterally, and the structure settles and may even tip out of vertical alignment. This kind of settlement, caused by the failure in bearing capacity of soil, is usually developed suddenly and rapidly. Its amount is not predictable and can not be allowed for in the design of foundations. Therefore, this kind of settlement will not be discussed here.

In the second type, a structure settles by virtue of the compressive stress and the accompanying strains developed in the soil by the load imposed on it without failure of the soil. This kind of settlement, caused by the reduction in volume of a soil mass resulting from the application of a foundation load and the accompanying compressive stress and strain, is called consolidation. This will be considered as the major part of settlement in this study.

Because the soil mass lies beneath a limited horizontal plane surface and extends to an infinite distance in all directions below that plane, it is considered as a semi-infinite solid. The transmission of surface load into the subsoil will produce vertical, horizontal, radial and shearing stresses within the soil mass. The volume change of soil mass may be caused by the combined effects of vertical consolidation due to vertical pressure and upward displacement due to lateral pressure and shearing stresses. But in the soil masses in the soil column directly under the loaded area (Fig. 1), the vertical stress is much greater than the other stresses. Therefore, the effect of the settlement component due to the vertical consolidation usually predominates in comparison to the others, except for soils such as very soft clays which are weak in shearing resistance and easily displaced like a viscous fluid. For that reason, settlement due to the volume changes in a soil mass resulting from the application of foundation loads and the accompanying compressive stress and strain may be assumed to be in the vertical direction only.

Based on this assumption, the settlement S in a soil column in a homogeneous soil layer of thickness H directly under a single load P, as shown in Figure 1, is equal to the sum of the vertical compressive strains in the successive horizontal layers of thickness dZ due to the vertical pressure transmitted by P:

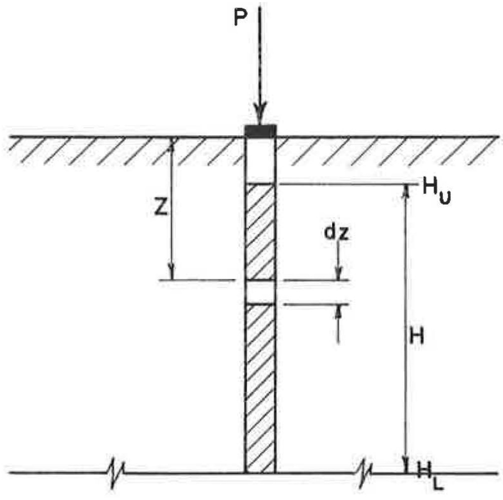


Figure 1. Soil column under single load.

$$S = \int_{H_u}^{H_\ell} \epsilon \, dZ \qquad (1)$$

in which

$\epsilon$ = unit strain for a layer dZ at depth Z below loaded area,
$H_u$ = upper boundary of soil layer, and
$H_\ell$ = lower boundary of soil layer.

Because soil is porous material containing a large proportion of void space, the strain due to volume change in soil actually corresponds to a decrease in void space, although there may be a negligible compression of water in the soil and of soil grains at their points of contact due to intergranular pressure. The unit strain $\epsilon$ may, therefore, be expressed in terms of the change of void ratio:

$$\epsilon = \frac{\Delta e}{1 + e} \qquad (2)$$

in which

e = original or known void ratio,
$\Delta e$ = change of void ratio, and
1 + e = total volume of soil.

The character of the change of void ratio for most soil in consolidation tests is such that the curve of void ratio vs pressure plotted on a semilogarithmic paper, as shown in Figure 2, is almost a straight line, and its slop $C_c$, the compression index, is a constant. By this relationship, the change of void ratio may be written in terms of pressure as:

$$\begin{aligned} \Delta e &= C_c \, \Delta \log P \\ &= C_c \, (\log T - \log B) \\ &= C_c \log \frac{T}{B} \\ &= C_c \log \left(\frac{B + V}{B}\right) \\ &= C \log \left(1 + \frac{V}{B}\right) \qquad (3) \end{aligned}$$

in which B is existing overburden pressure before surface load is applied and T is total pressure equal to B plus the additional vertical pressure transmitted by P.

The magnitude of vertical pressure transmitted by the surface load depends on the relative location of the point of loading and the stressed point in the subsoil, and is governed by the transmission factor, A:

$$V = A \cdot P \qquad (4a)$$

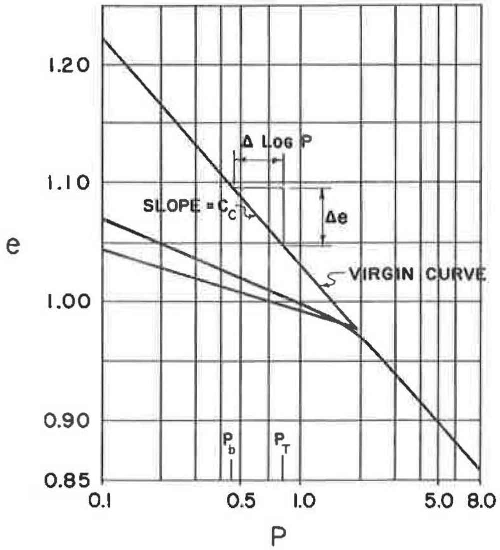


Figure 2. Idealized void ratio vs pressure curve.

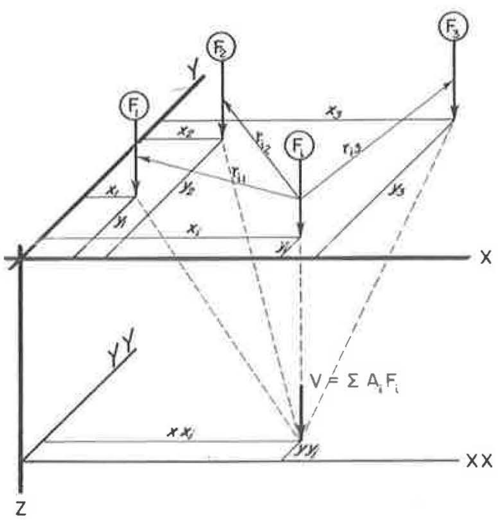


Figure 3. Transmission of surface load.

Since Eq. 4a considers one point load only, if there are n point loads at the surface, as shown in Figure 3, the vertical pressure is then equal to

$$V = \sum_{i=1}^{n} A_i P_i = 1 \ldots \qquad (4b)$$

By combining Eq. 1 with the relationships expressed by Eqs. 2, 3, 4a, and 4b, we obtain for settlement due to one surface load:

$$S = \int_{H_u}^{H_\ell} \frac{C_c}{1+e} \log \left(1 + \frac{A \cdot P}{B}\right) dZ \qquad (5a)$$

or for settlement due to a combination of surface loads:

$$S = \int_{H_u}^{H_\ell} \frac{C_c}{1+e} \times \log \left(1 + \frac{\sum_{i=1}^{n} A_i P_i}{B}\right) dZ \qquad (5b)$$

Because of the multiplicity of loads and the nonlinear character of the transmitting factor, numerical methods have to be used to evaluate the equations and the modified Euler method was adopted. The basic assumption of this method is the approximation that the unit strain at the mid-depth of a small increment of thickness ΔZ is the mean value of unit strain for that layer. Eq. 4b is then rewritten as

$$S_j = \sum_{k=1}^{\ell} \frac{C_c}{1+e} \times \log \left(1 + \frac{\sum_{i=1}^{n} A_{jki} P_i}{B_{jk}}\right) \Delta Z \qquad (6)$$

in which

$j$ = number of calculating point,
$k$ = number of increment, and
$i$ = number of point load.

## TRANSMITTING FACTORS

Some equations developed to determine the transmitting factors are as follows:

1. Boussinesq equation for a point load applied to a homogeneous isotropic semi-infinite elastic mass:

$$A = \frac{3}{2\pi} Z^3 \left(r^2 + Z^2\right)^{-5/2} \tag{7}$$

2. Westergaard equation for point load applied to a horizontal layer of an elastic mass infinitely rigid in the horizontal direction:

$$A = \frac{1}{2\pi}\left[1 + 2\left(\frac{r}{Z}\right)^2\right]^{-3/2} \tag{8}$$

3. Steinbrenner equations for a uniformly loaded area with unit load P for rectangular loading area (calculating point at center):

$$A = \frac{2}{\pi}\left\{\arc\tan\left[\frac{b'}{Z}\;\frac{a'(a'^2 + b'^2) - 2a'Z(R' - Z)}{(a'^2 + b'^2)(R' - Z) - Z(R' - Z)^2}\right] + \frac{a'b'Z(R'^2 + Z^2)}{(b'^2 + Z^2)(a'^2 + Z^2)R'}\right\} \tag{9}$$

in which

$a'$ = 1/2 of width a,
$b'$ = 1/2 of length b, and
$R' = \sqrt{a^2 + b^2 + c^2}$;

and for circular loading area (calculating point at center):

$$A = 1 - \left(\frac{1}{1 + \frac{R^2}{Z^2}}\right)^{3/2} \tag{10}$$

in which R is radius of circular section. These equations, when properly applied, will serve reasonably well to determine the vertical pressure in the subsoil. A mistake in the selection of the transmitting factor will cause a great error in the computation. For example, when the calculating point is directly under the loading point, for which the horizontal distance r is equal to zero in both the Boussinesq and Westergaard equations, the transmitting factors are infinity as the depth Z approaches zero. This may be explained by the fact that a point load is not really a point load but a load distributed over a certain area. Therefore, when the calculating point lies directly under the loading point, the transmitting factor of a distributed surface load should be taken into consideration instead of a point load.

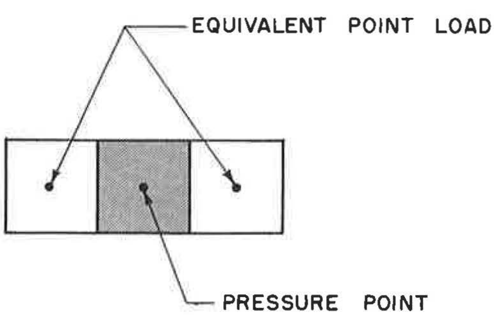


Figure 4. Assumed loading.

Because the transmitting factor equation for a rectangular loaded area is rather complicated, the authors suggest the use of the equation for a circular section having an equivalent area of the rectangular section. If the width and length are greatly different, we can divide the area into several approximately square sections, as shown in Figure 4, and consider the section above the calculating point as a distributed loaded area and the others as point loads. The difference between the transmitting factor for a square section and that for a circular section is very small.

The final expression for the transmitting factor shown in Eq. 6 is written as follows for a point load:

$$A_{jki} = \frac{3}{2\pi} Z_{jk}^{3} \left[1 + \left(\frac{r_{jk}}{Z_{jk}}\right)^{2}\right]^{-5/2} \quad \text{(Boussinesq)} \tag{11}$$

or

$$A_{jki} = \frac{1}{2\pi} \left[1 + 2\left(\frac{r_{ji}}{Z_{jk}}\right)^{2}\right]^{-3/2} \quad \text{(Westergaard)} \tag{12}$$

when $r_{ji} \neq 0$; and for a distributed load:

$$A_{jki} = \left[1 - \left(\frac{1}{1 + \left(\frac{R_{ji}}{Z_{jk}}\right)^{2}}\right)^{3/2}\right] \tag{13}$$

as $R_{ji} = 0$

## EXISTING OVERBURDEN PRESSURE

The existing overburden pressure at any point of the subsoil may be expressed in the form

$$B_{jk} = B_{j(k-1)} + G\Delta Z \tag{14}$$

in which

$B_{j\,(k-1)}$ = existing pressure above the increment layer dZ, and
$G$ = effective unit weight of subsoil.

It should be noted that the existing overburden pressure at the loading surface is not zero but is equal to the weight of soil excavated. The effective unit weight is equal to the submerged weight if the subsoil is under the water table.

## SIGNIFICANT ACTING PRESSURE

The vertical pressure transmitted down to the horizontal planes in the subsoil, as shown in Figure 5, would normally spread out with depth. Thus, the increase of the horizontal distance from the point of application of load will result in decreased magnitude of the vertical pressure. The increment of vertical pressure at the point such that r/Z is greater than 2 is very small. Therefore, the surface loads located beyond the horizontal distances greater than 2Z need not be taken into account.

If the subsoil layer extends to a great depth, the compressive strains will vanish when the vertical pressure is very small compared to the overburden pressure. It is, therefore, generally safe to assume that stress due to boundary loading is no longer significant in regard to settlement when it is less than 10 percent of the existing overburden pressure, except in formations which have a presumptive bearing capacity of zero.

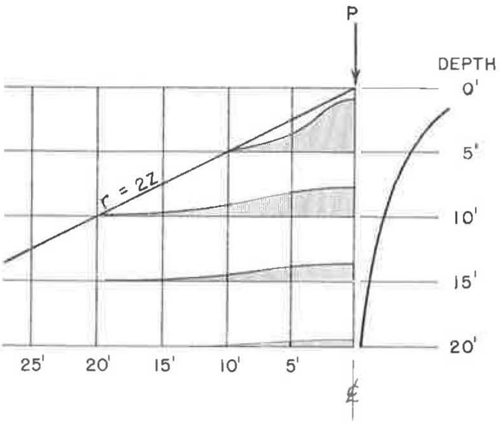


Figure 5. Vertical stress distribution on horizontal planes.

TABLE 1

LOADING DATA FOR COMPUTER

| NN | F (lb) | P (psf) | X (ft) | Y (ft) |
|---|---|---|---|---|
| 1 | 48000 | 3000 | 0 | 20 |
| 2 | 48000 | 3000 | 0 | 20 |
| 3 | 56000 | 3500 | 20 | 20 |
| 4 | 56000 | 3500 | 20 | 0 |

## DEVELOPMENT OF COMPUTER PROGRAM

Because the magnitude of settlement is dependent on the magnitude of surface load, its point location, and the properties of the soil between the surface and the point at which the settlement is required, all these data must be listed before proceeding with the program.

The first step is to obtain the loading data, which include the magnitude of point loads, the average unit contact pressure of point load foundations, and the location of the point loads. Each individual foundation is treated as a single point if its shape is approximately square or circular or if the loaded area is not too large; otherwise, the loaded area is divided into shapes as described previously. As soon as the point loads are decided, the relative location of point loads may be determined by laying out rectangular coordinates on horizontal planes, as shown in Figure 3. The coordinate values are assigned to each point load to signify the position of the point from left to right and from top to bottom. For convenience in the work, a tabular form is set up as shown in Table 1.

The second step is to decide on the points at which settlement computations are required. The procedure for locating these points is the same as for the point loads. The soil properties in the different layers, such as void ratio, unit effective weight, compression index and the upper

TABLE 2

SOIL DATA FOR COMPUTER

| MM | XX (ft) | YY (ft) | BS (psf) | LN | G (pcf) | E | C | $H_u$ (ft) | $H_\ell$ (ft) |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 0.0 | 20 | 500 | 1 | 100 | 0.95 | 0.05 | 0 | 6 |
| | | | | 2 | 110 | 0.81 | 0.04 | 6 | 12 |
| | | | | 3 | 60 | 0.81 | 0.04 | 12 | 18 |
| | | | | 4 | 50 | 0.90 | 0.05 | 18 | 26 |
| | | | | 5 | 42 | 0.86 | 0.05 | 26 | 32 |
| 2 | 0.0 | 0 | 0 | 1 | 100 | 0.95 | 0.05 | 0 | 6 |
| | | | | 2 | 110 | 0.81 | 0.04 | 6 | 12 |
| | | | | 3 | 60 | 0.81 | 0.04 | 12 | 18 |
| | | | | 4 | 50 | 0.90 | 0.05 | 18 | 26 |
| | | | | 5 | 42 | 0.86 | 0.05 | 26 | 32 |

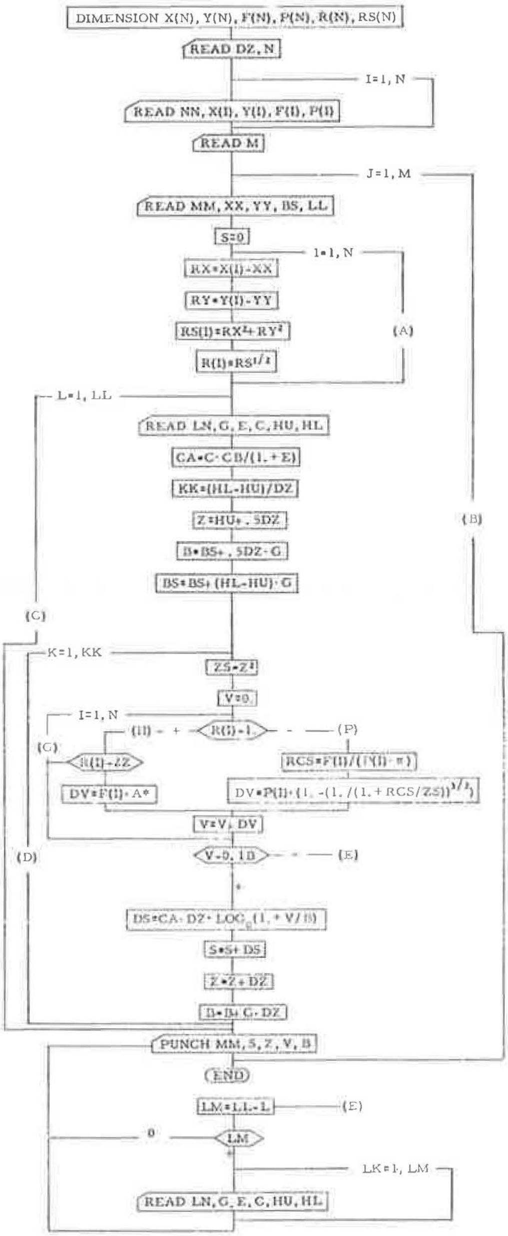

Figure 6. Flow diagram.

and lower boundary of the soil layer, should be provided in proper order, layer to layer, from the surface to the required depth. These may be listed as shown in Table 2.

## FLOW DIAGRAM

Using Eq. 6, flow diagram is constructed as shown in Figure 6. The specific FORTRAN language program is given in Appendix B and the symbols used are defined in Appendix A. The procedures of the program are shown in the flow diagram, but some of the statements, which might be easily confused, are explained as follows.

In Section D when V - 0.1B is negative, the settlement at this point is not significant and there is no need of further calculations; the program then will be shifted to Section E where the computer checks how many data cards for this calculation point are left, reads all of them, then jumps to the next calculation point.

In Section G when R(I) - 1.0 is negative, R(I) is approaching zero. The point load is treated as a distributed loading area, and the program goes to Section P. Otherwise the progression is to Section H.

In Section H when R(I) - 2Z is positive, the load on the surface has no effect on this point and there is no need to calculate the vertical pressure at this point. The program goes back to Section G.

## SAMPLE PROBLEMS

The settlement pattern for a building in Brazil was computed using the program developed and an IBM 1620 computer. Details of the solution are given in Appendix C. The problem involved 33 loaded points on a soil having two layers with a total depth of 55 ft. Settlements were computed for each 1-ft increment of depth. The computer time used was 2.5 hr.

Another problem was solved for 16 loaded points and a single 30-ft layer of soil using both a 1- and a 0.1-ft increment for calculating settlements. The computer time used was 10 min for the 1-ft increment and 100 min for the 0.1-ft increment. The difference between the total settlements in each case was only about 5 percent.

## CONCLUSIONS

The settlements obtained by the computer solution have been compared with some observed settlements and are found to be in reasonable agreement with the actual settlements (Appendix C). The computer solution avoids computation errors which might occur if these lengthy computations are made in another manner. The solution can be used for complex or simple structures.

The time required to execute the program is dependent on the computer speed, the number of loads, thickness of soil layer and the increment dZ selected. However, the computer solution compares very favorably in cost, accuracy and speed with any of the other methods that might be used. The greater the complexity of structures, the better is the comparison.

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# *Appendix A*

## NOMENCLATURE

A = transmission factor,

B = existing overburden pressure before surface load is applied,

C = compression index of soil in flow chart,

E = void ratio in flow chart,

F = equivalent concentrated point load,

G = effective unit weight of soil,

K = sequential number for increment DZ,

M = total number of calculation points,

N = total number of point loads,

P = average unit contact pressure of point load,

R = horizontal radial distance between calculation point and loading point,

S = total settlement,

T = total pressure in soil after surface load is applied,

V = vertical pressure transmitted from surface load to a point in the soil,

X = distance of point load from Y axis,

Y = distance of point load from X axis,

Z = depth from contact surface to calculation point,

BS = overburden pressure at upper boundary of soil layer,

CB = constant equal to 12· $\log_e/\log_{10}$ (5.211534 for inch system),
DS = increment of settlement for thickness DZ,
DV = increment of vertical pressure from a point load,
DZ = increment of thickness of soil layer,
HL = depth of lower boundary of soil layer for flow chart,
HU = depth of upper boundary of soil layer for flow chart,
KK = number of increments of thickness DZ,
LL = total number of soil layers under calculation point,
LM = number of data cards in which data is beyond significant settlement,
LN = sequential number for soil layers,
MM = sequential number for calculation points,
NN = subscript for point load data,
RCS = square of radius of equivalent circular loaded area,
RS = square of R,
XX = distance of calculation point from YY axis,
YY = distance of calculation point from XX axis,
$C_c$ = compression index of soil,
$H_u$ = upper boundary of soil layer,
$H_\ell$ = lower boundary of soil layer,
e = original void ratio of soil,
$\Delta e$ = change of void ratio,
1 + e = total volume of soil,
r = horizontal distance between loading point and calculation point, and
$\epsilon$ = unit strain for layer dZ at depth Z below loaded area.

## *Appendix B*

### FORTRAN PROGRAM FOR IBM 1620 COMPUTER

```
      DIMENSION X(100), Y(100), P(100), F(100), R(100), RS(100)
      READ, DZ, N
      DO 2I=1, N
    2 READ, NN F(I), P(I), X(I), Y(I)
    3 READ, M
      DO 10 J=1, M
      READ, MM, XX, YY, B, LL
      DO 11 I=1, N
      RX=X(I)-XX
      RY=Y(I)-YY
```

```
      RS(I)=RX*RX+RY*RY
      R(I)=RS(I)**.5
11    CONTINUE
      DO 20 L=1, LL
      READ,  LN, G, E, C, HA, HB
      CA=5.211534*C/(1.+E)
      KK=(HL-HU)/DZ
      Z=HA+.5*DZ
      B=BS+.5*G*DZ
      BS=BS+(HL-HU)*G
      DO 30 K-1, KK
      ZS=Z*Z
      V=0.0
      DO 40 I=1, N
      IF (R(I)-1.0) 41, 41, 42
42    RCS=F(I)/(P(I)*3.1416)
      DV=P(I)*(1./(1.+RCS/ZS))**1.5)
      V=V+DV
      GO TO 40
41    IF (R(I)-2.*Z)43, 40, 40
43    DP=.477*F(I)/(ZS*(1.+RS(I)/ZS)**2.5)
      V=V=+DV
40    CONTINUE
      IF (PV-.1*B) 13, 31, 31
31    DS=CA*LOG(1.+V/B)*DZ
      S=S+DS
      Z=Z+DZ
      B=B+G*DZ
30    CONTINUE
20    CONTINUE
12    PUNCH,  MM, S, Z, V, B
10    CONTINUED
13    LM=LL-L
      IF (LM) 12, 12
      DO 15 KN=1, KK
      READ,  G, E, C, HA, HB
15    CONTINUE
      GO TO 12
      END
```

# *Appendix C*

## SAMPLE PROBLEM

The settlement of a building at Santos, Brazil, has been calculated using the program presented in Appendix B. The building has a total load of 9, 514 tons carried on strip footings as shown in Figure 7. The footings were placed 8 ft below the surface and were designed for a soil pressure of 3 tons/sq ft. For convenience in calculating the stresses in the soil, an equivalent loading plan as shown in Figure 7 was devised. The loading data for this plan are listed in Table 3.

The strata below the foundations are shown in Figure 7. The water level is 3 ft below the ground surface. The average liquid limit for the clayey sand is 35 percent. From this information, we assume the needed data such as void ratio, effective unit weight and the compression index, as listed in Table 4.

The loading and soil properties were determined and placed in the computer. The settlements calculated are shown in Figure 8. Table 5 gives these settlements along with those actually measured at the building site.

TABLE 3

LOADING DATA

| NN | X (ft) | Y (ft) | F (lb) | P(psf) |
|---|---|---|---|---|
| 1 | 5 | 37 | 600000 | 6000 |
| 2 | 15 | 37 | 600000 | 6000 |
| 3 | 25 | 37 | 600000 | 6000 |
| 4 | 35 | 37 | 600000 | 6000 |
| 5 | 45 | 37 | 600000 | 6000 |
| 6 | 55 | 37 | 600000 | 6000 |
| 7 | 65 | 37 | 600000 | 6000 |
| 8 | 75 | 37 | 600000 | 6000 |
| 9 | 85 | 37 | 600000 | 6000 |
| 10 | 95 | 37 | 600000 | 6000 |
| 11 | 105 | 37 | 600000 | 6000 |
| 12 | 5 | 21 | 600000 | 6000 |
| 13 | 15 | 21 | 600000 | 6000 |
| 14 | 25 | 21 | 600000 | 6000 |
| 15 | 35 | 21 | 600000 | 6000 |
| 16 | 45 | 21 | 600000 | 6000 |
| 17 | 55 | 21 | 600000 | 6000 |
| 18 | 65 | 21 | 600000 | 6000 |
| 19 | 75 | 21 | 600000 | 6000 |
| 20 | 85 | 21 | 600000 | 6000 |
| 21 | 95 | 21 | 600000 | 6000 |
| 22 | 105 | 21 | 600000 | 6000 |
| 23 | 5 | 5 | 600000 | 6000 |
| 24 | 15 | 5 | 600000 | 6000 |
| 25 | 25 | 5 | 600000 | 6000 |
| 26 | 35 | 5 | 600000 | 6000 |
| 27 | 45 | 5 | 600000 | 6000 |
| 28 | 55 | 5 | 600000 | 6000 |
| 29 | 65 | 5 | 600000 | 6000 |
| 30 | 75 | 5 | 600000 | 6000 |
| 31 | 85 | 5 | 600000 | 6000 |
| 32 | 95 | 5 | 600000 | 6000 |
| 33 | 105 | 5 | 600000 | 6000 |

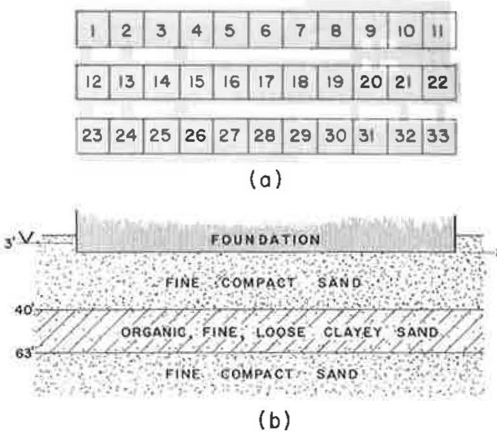


Figure 7. Sample problem: (a) assumed loading on soil, and (b) soil profile.

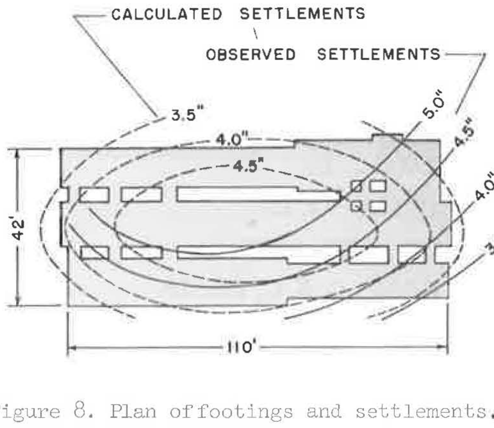


Figure 8. Plan of footings and settlements.

TABLE 4

SOIL DATA

| MM | XX | YY | BS | LN | G | E | C | HU | HL |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 5 | 37 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 2 | 15 | 37 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | -.16 | 32 | 55 |
| 3 | 25 | 37 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 4 | 35 | 37 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 5 | 45 | 37 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 6 | 55 | 37 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 7 | 65 | 37 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 8 | 75 | 37 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 9 | 85 | 37 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 10 | 95 | 37 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 11 | 105 | 37 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 12 | 5 | 21 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 13 | 15 | 21 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 14 | 25 | 21 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 15 | 35 | 21 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 16 | 45 | 21 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 17 | 55 | 21 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 18 | 65 | 21 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 19 | 75 | 21 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 20 | 85 | 21 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 21 | 95 | 21 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 22 | 105 | 21 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 23 | 5 | 5 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 24 | 15 | 5 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 25 | 25 | 5 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 26 | 35 | 5 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 27 | 45 | 5 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 28 | 55 | 5 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 29 | 65 | 5 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 30 | 75 | 5 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 31 | 85 | 5 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 32 | 95 | 5 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |
| 33 | 105 | 5 | 680 | 1 | 70 | 0.5 | 0.01 | 0 | 32 |
| | | | | 2 | 25 | 1.5 | 0.16 | 32 | 55 |

TABLE 5

SETTLEMENT

| Pt. No. MM | Calc. Set. (in.) | Pt. No. MM | Calc. Set. (in.) |
|---|---|---|---|
| 1 | 3.344 | 18 | 5.133 |
| 2 | 3.947 | 19 | 5.029 |
| 3 | 4.302 | 20 | 4.813 |
| 4 | 4.500[a] | 21 | 4.418 |
| 5 | 4.600 | 22 | 3.741 |
| 6 | 4.626 | 23 | 3.344 |
| 7 | 4.600 | 24 | 3.947 |
| 8 | 4.500 | 25 | 4.302 |
| 9 | 4.302 | 26 | 4.500 |
| 10 | 3.947 | 27 | 4.600 |
| 11 | 3.344 | 28 | 4.626 |
| 12 | 3.741 | 29 | 4.600 |
| 13 | 4.418 | 30 | 4.500 |
| 14 | 4.813 | 31 | 4.302 |
| 15 | 5.029 | 32 | 3.947 |
| 16 | 5.133 | 33 | 3.344[c] |
| 17 | 5.164[b] | | |

[a]Observed settlement = 5.25 in.
[b]Observed settlement = 6.75 in.
[c]Observed settlement = 3.30 in.