THE UNIVERSITY OF MANITOBA

PRESSURE DISTRIBUTION
UNDER AN
EXTENDED RING
SILO FOUNDATION

by

Murray Drew Vanderpont

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ABSTRACT

Pressure Distribution under an
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An analysis of pressure distribution under the extended ring foundation commonly used for concrete tower silos was conducted. The proposed method of modelling the foundation was to treat the ring footing and silo floor area as separate load transfer media. Classic silo foundation analysis, based on the assumption that the entire weight of the silo and contents are uniformly distributed over the whole area encompassed by the outer diameter of the ring footing was used as a basis of comparison.

Load conditions were evaluated separately for the ring footing and silo floor area. A reference contact pressure for evaluating safety factor against bearing capacity failure was determined. A computer program was developed to calculate contact pressure under both classical and the proposed theory and also to define stress increase in the supporting soil. The proposed method was applied to 30 silos which had been analyzed under the classical theory.

Results indicate that classic theory underestimates the maximum contact pressure, found under the heel of the ring footing, by an average of 35%. Application of the proposed theory significantly alters the pattern and intensity of stress distribution in the supporting soil.

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Chapter I

INTRODUCTION

Concrete tower silos are a familiar sight in most of the livestock producing regions of Canada and the United States. It has been estimated by Bozozuk (1979 a) that over 3,000 silos are constructed annually in Canada. These bullet shaped structures have proved themselves to be an efficient and economical means for storing and handling silage on livestock operations. In recent years, the trend has been towards larger and larger silos. They are now being constructed with diameters exceeding 9.000 m and heights exceeding 32.000 m. The higher loads which accompany these larger silos have led to increasing concern over the performance of the structure's foundation with respect to settlement, tilting, and stability.

In the past, tower silo footings have been built with almost no attention to geotechnical considerations. Design of the footings based on the experience of the contractor has generally been the norm. The fact that most silos are performing well is a tribute to the industry. However, foundation design becomes more critical as size increases, not only because load increases, but the depth to which significant stressing occurs also increases. The result of this combination is greater potential for foundation problems. Design of the foundations for a structure weighing as much as 1100 tonnes, as high as a 10 story building, and costing $100,000 should be engineered on the basis of geotechnical principles. Inadequate foundations often result in excess settlements which can lead to problems with mechanical loaders and unloaders. Tilt associated with excess non-uniform settlement leads to underutilization of the silo with an accompanying economic penalty. Catastrophic bearing capacity failure is the ultimate danger associated with inadequately designed foundations.

A common footing used for tower silos is a reinforced concrete extended ring footing. This type of footing has the silo superstructure supported on a ring or doughnut shaped footing. The term 'extended' is used because the silo walls contact the footing close to the inner diameter of the ring. This leaves a large portion of the footing extending out from the walls of the silo. Extended ring footings are used as a method of increasing the bearing area of the foundation beyond the walls of the silo without using concrete under the floor. This simplifies drainage provisions via reduced formwork, and also reduces concrete costs. The extra wide base is further intended to provide enhanced stability against overturning.

Present procedures for determining contact pressures for bearing capacity calculations make the assumption that the total load of the structure and contents is uniformly distributed over the total area circumscribed by the outer diameter of the ring. This includes both the area of ring/soil interface and the area enclosed by the ring or the floor/soil interface. This assumption is used when computing the safety factor against bearing capacity failure. For engineering design, the theoretical load carrying ability of the foundation should normally be at least three times the applied load. In most cases, this value provides structural stability and acceptable settlement for tower silos.

An alternative method of considering the contact pressure under an extended ring silo foundation is to treat the concrete ring and the silo floor as separate and distinct footings. This composite footing approach allows the foundation loading to be described as a line load provided by the walls and carried by the ring, and a uniformly distributed load provided by the silage mass and carried by the area inside the ring.

In light of the increasing concern over the performance of the large tower silo foundations, the objectives of this study are to re-examine the distribution of contact pressure under the extended ring footing and to analytically determine the pressure distribution in the soil under the footing. This includes the development of a BASIC program to calculate foundation contact pressures for a given silo and produce resultant stress increase isobars for the underlying soil, under conditions of uniform loading. Using this program a comparison between the two methods of analysis, that is the single round footing approach and the composite footing approach, will be made.

Chapter II

REVIEW OF LITERATURE

2.1 Contact Pressure

For the purpose of determining the contact pressure under an extended ring silo foundation, the total weight of the structure and contents is commonly considered to be uniformly distributed over a circular area described by the outer diameter of the ring (Bozozuk 1974). This approach has been accepted by most workers (Turnbull 1979, Becker and Lo 1979, Morin and Bozozuk 1983). The validity of this approach has not been supported by research to date.

On a full scale instrumented silo study conducted by Becker and Lo (1979) pressure distribution under a doughnut foundation was investigated. Output from pressure cells arrayed under the 9.100 m diameter by 21.900 m high tower indicated average contact pressure below the ring footing and the silo floor increased linearly and by equal amounts for the first third of the filling height. As more silage was added, contact stress increased faster under the ring footing than under the silage. It was found that after the silo was filled to two-thirds height, a distinctly non-uniform contact pressure distribution existed across the entire base of the foundation. At full height the minimum pressure under the floor was less than half the pressure under the ring.

Results close to the Becker and Lo (1979) study were obtained by Bozozuk (1979 b) from a 7.320 m diameter by 24.280 m high similarly instrumented silo. When the structure was filled to 85% capacity, the proportion of silage load transfer to the extended ring foundation was 50% +/- 3%. The average pressure under the silage was 45% greater than the average pressure under the footing. Contact pressure measured under the inner edge of the ring was twice as great as that measured under the outer edge.

Because of the eccentric position of the wall on extended ring footings, load transmitted by the wall to the footing is distributed non-uniformly to the underlying soil (Bozozuk 1974). From his instrumented silo study, Bozozuk found that contact pressure is normally higher at the inner edge of the ring. The difference was reported to be as much as 50% when the silo was first loaded (Bozozuk 1979 b). Because of the uneven contact pressure due to load eccentricity, local overstressing under the heel, or inner radius, of the ring can occur which could cause the footing to tip, twist, deform, or crack into pieces (Bozozuk 1974). When a foundation carries an eccentric load, it tends to tilt towards the eccentricity. The contact pressure is then taken to decrease linearly towards the toe, or outer radius, from a maximum at the heel (Meyerhof 1953). Meyerhof also suggested that the load transmitted by the footing to the soil be taken to act on an effective footing width equal to the actual width minus twice the eccentricity.

From Bozozuk's instrumented silo study, it was found that the differential settlement of the ring, on the supporting marine clay, from heel to toe was 15 mm to 20 mm. The maximum difference was 32 mm. He contended that a footing with such large deformation cannot supply uniform support. He suggested that the ring "beds in" until more or less uniform support is achieved (Bozozuk 1979 b).

A survey of 108 concrete tower silos in Quebec assessed the performance of the foundations with respect to settlement, tilt, and safety factor against bearing capacity failure (Morin and Bozozuk 1983). All of the silos were supported on extended ring footings. Uniform pressure distribution across the whole base of the foundation was also assumed. At 28 of the sites, detailed investigations were conducted. It was found that seven of these silos, with a mean safety factor of 1.94, performed unacceptably with vertical settlements between 150 mm and 300 mm and tilts of 1.0° to 1.5°. This amount of movement severely hampered operation of the silo. An additional four silos, with a mean safety factor of 1.63, underwent settlements in excess of 300 mm and tilted in excess of 1.5°. This amount of movement left the the structures virtually unuseable. Of the 108 silos included in the survey, 11 or 10% had foundations considered unacceptable by the owners due to settlement and tilt resulting in interference with silo operations. Based on the results of the 28 sites at which detailed investigations were made, the authors recommended a safety factor against bearing capacity failure of 3.0 to ensure acceptable foundation performance.

Most catastrophic bearing capacity failures of tower silos occur the first time that the silo is filled. Because soil shear strength decreases with increases in excess porewater pressure, the danger of a bearing capacity failure is at a maximum when the silo is first filled to capacity (Bozozuk 1979 b). This is also the time when non-uniformity of contact pressure is most evident.

2.2 Contact Pressure Distribution

The purpose of accurately determining the contact pressure distribution under any type of footing is two-fold. Firstly, the safety factor against bearing capacity failure is equal to the ratio of the calculated bearing capacity of the soil to the applied load intensity. Secondly, settlement is directly related to stress levels in the supporting soil which is in turn a function of the intensity and distribution of the applied contact stresses. A knowledge of contact pressure distribution is essential if the foundation is to be designed to meet the requirements of carrying the applied load without shear failure and the settlement being acceptable for that structure (Bowles 1982).

2.3 Subgrade Stresses

Stresses and strains in the soil underlying a footing depend on the stress/strain relationships of that soil. Precision analysis is made difficult because these relationships are non-linear (Craig 1978). To facilitate soil stress analysis, four major assumptions are often made. The soil mass is assumed to be semi-infinite, homogenous, isotropic and linearly elastic (Graham 1977). These assumptions are not too restrictive when stresses are limited to about 1/3 the ultimate bearing capacity of the soil. At this loading, the condition of a linearly elastic stress/strain relationship in the soil mass is not unreasonable in many cases. By making use of these simplifying assumptions, researchers have developed models which attempt to predict stresses in a soil mass due to footing pressure.

2.4 Spread Method

Probably the simplest method, both theoretically and practically of estimating subgrade stresses, is the 2:1 slope method. This approach makes use of the fact that stress is equal to force divided by area and the idea that the load on a footing is effectively supported by an increasingly larger area of soil as depth below the footing increases. The spread method merely specifies the slope to use when calculating the equivalent area of support. This method is an approximation. However, it compares favourably with more analytical methods to a depth of about four times the footing width (Bowles 1982).

2.5 Analytic Methods

The two more mathematically orientated methods of estimating soil pressures at various points in the soil stratum are the Boussinesq and the Westergaard theories. Both of these theories are based on elastic methods (Bowles 1982). In 1885, Boussinesq developed an equation to describe stresses due to a surface point load inside a semi-infinite, homogenous, isotropic mass to which Hooke's Law applies. The equation models stress increase as a function of magnitude of loading and the vertical and horizontal distances between point of loading and point of interest. The stresses due to uniformly distributed surface loads can be determined by integration from the point load solution. Stress increase at a point due to more than one load can also be found by superposition (Craig 1978). When the soil mass consists of many layers of finer and coarser materials, such as varved clay, the Boussinesq solution becomes unreliable (Bowles 1982). In 1938, Westergaard presented a mathematical model for this situation. It incorporated the same parameters as Boussinesq and also included Poisson's ratio. In comparing the two models (Bowles 1982) found that:

i) The Boussinesq equation gives larger stress intensities close to the footing.

ii) At a radial distance to depth ratio of 1.8, stress intensities are predicted to be approximately equal using either theory.

iii) At a radial distance to depth ratios of greater than 1.8, the Westergaard equation gives larger stress intensities.

In 1935, Newmark developed an influence chart, based on Boussinesq, from which the stress increment at any point due to a load on the surface could be graphically determined. The chart is based on the equation :

$$r/z = ((1-q/q_o)^{2/3}-1)^{1/2) \qquad 2.1$$

where r = radius of a uniformly loaded circular area

z = depth below centre of circular area

q = pressure at a point at depth z below centre of circular area

q_o = surface loading intensity

The r/z ratio can be thought of as the relative size of a circular bearing area which, when loaded, gives an unique pressure ratio q/q_o on the element of soil at depth z in the stratum. This result was used by Newmark to draw a series of concentric circles which represent the size of footing required to obtain a certain stress increase. Newmark's influence chart can be used with footings of any size or shape. A similar influence chart, adapted for layered soils, has also been developed for Westergaard's theory (Bowles 1982). This chart is based on the equation:

$$r/z = + (a/(1-q/q_o)-a)^{1/2} \qquad 2.2$$

where $a = (1-2u) / (2-2u)$

u = Poisson's ratio

2.6 Pressure Bulbs

The concept of the pressure bulb has grown out of the use of the influence chart (Bowles 1982). Pressure bulbs, or isobars, have been constructed with the influence chart used to find pressure intensities at various locations beneath and outside the footing prism. Today, the easiest way to solve for pressure bulbs is by numerical integration with the digital computer (Bowles 1982). Also, at advanced levels, finite element techniques using more realistic stress/strain properties and anisotropic or non-homogeneous conditions can be used to determine stresses and displacements in the soil mass (Craig 1978). However, finite element methods require more sophisticated computers with large amounts of memory space, whereas the other methods are applicable to personal computers.

2.7 Bearing Capacity

Ultimate bearing capacity is defined as the stress level at which a foundation will experience excessive sink and tilt, and will cause soil heaving. This movement will continue until the foundation reaches a redistributed equilibrium stress state where bearing capacity is high enough to prevent further movement (Tomlinson 1975). The method for determining the bearing capacity of soil has been developed from plasticity theory for an infinite strip, and then modified for squares, rectangles, and circles (Bowles 1982). The general bearing capacity equation is comprised of three terms which account for soil type, footing geometry, and surcharge (Sowers 1979). A widely used equation was presented by Terzaghi in 1943. His equation for net ultimate bearing capacity takes the form of (Terzaghi and Peck 1967):

$$q_d = cN_c + UD_f(N_q) + UBN_u/2 \qquad 2.3$$

q_d = net ultimate bearing capacity (kPa)

where c = undrained cohesion of soil (kN/m^2)

N_c, N_q, N_u = bearing capacity factors (dimensionless)

D_f = distance from grade to base of footing (m)

U = density of soil below foundation level (kN/m^3)

B = width of footing (m)

For shallow footings on clay, the cN_c term dominates (Bozozuk 1974). The procedure is applicable to foundations on sands, gravels, silts, clays and intermediate soils. Evidence collected from over 30 years of use testifies to the soundness of this approach (Sowers 1979). Shape, depth, and inclination factors are also used in conjunction with the general bearing capacity equation (Bowles 1982).

2.8 Distribution of Silage Loads

It is generally accepted that the vertical load in a silo is carried partially by the walls and partially the floor. Tests of model silos reveal that the entire vertical load is carried by the floor to a silage depth of one half the silo diameter. As silage depth increases, the walls take on an increasing proportion of the load. At a height to diameter ratio of three, total wall and floor load is equal. After this point, all further loading is carried by the walls (Negi et al. 1977). Using a finite element modeling technique, it was determined that the percentage of the silage load carried by the wall increases as the friction coefficient and height to diameter ratio increases. At a height to diameter ratio of three, 62% to 63% of the load would be carried by the wall (Jofriet and Czajkowski 1980). From an instrumented silo at Hammond, Ontario (Bozozuk 1979 b) it was found that the load was proportioned at 50% +/- 7.5% between floor and walls for height to diameter ratios of approximately 1.50 to 3.00.

An equation for estimating the proportion of the silage load supported by wall friction (Turnbull et al. 1979) is published in the CANADIAN FARM BUILDING CODE 1977. It is of the form:

$$F = Wh / 4.7D \; (1 - h/14.1D) \qquad 2.4$$

where F = vertical silage friction (kN)

W = total silage mass in silo (kg)

h = depth of silage (m)

D = silo diameter (m)

The equation is based on the assumption that the wall load equals the mass of silage above a right circular cone which has a base described by the silo floor, and a height equal to 4.7 times the diameter. This equation gives results similar to Negi et al. (1977) except for wider silos where higher wall loads are predicted. This deviation of equation 2.4 from the findings of Negi et al. (1977) is considered acceptable because it results in a conservative estimate of wall load, but may underestimate the floor load (Turnbull et al. 1979).

Chapter III

FOUNDATION ANALYSIS

3.1 Loading

There are three sources of load acting on the foundation soil of a tower silo. They are due to the silage mass, the structural mass, and the soil overburden to the footing level. It has been reported by Becker and Lo (1979) that wind load has no measurable effect on contact pressure. Construction details of a typical extended ring tower silo are shown in figure 3.1.

The silage mass can be found by the simple relation:

$$S = V_S \times M_S \qquad 3.1$$

where S = silage mass (kg)

V_S = volume of silage (m^3)

M_S = mass density of silage (kg/m^3)

Values for silage density at 70 % wet basis moisture content can be found in the Canadian Farm Building Code (1983). The density values vary with depth of silage, and tend to be somewhat conservative. They are, however, useful for design purposes.

The load due to the structure can be determined quite accurately by summing the mass of the structure and the mass of any mechanical equipment such as an unloader. The mass of the structure must include the roof, walls and footing because they all contribute significant load to the founding soil.

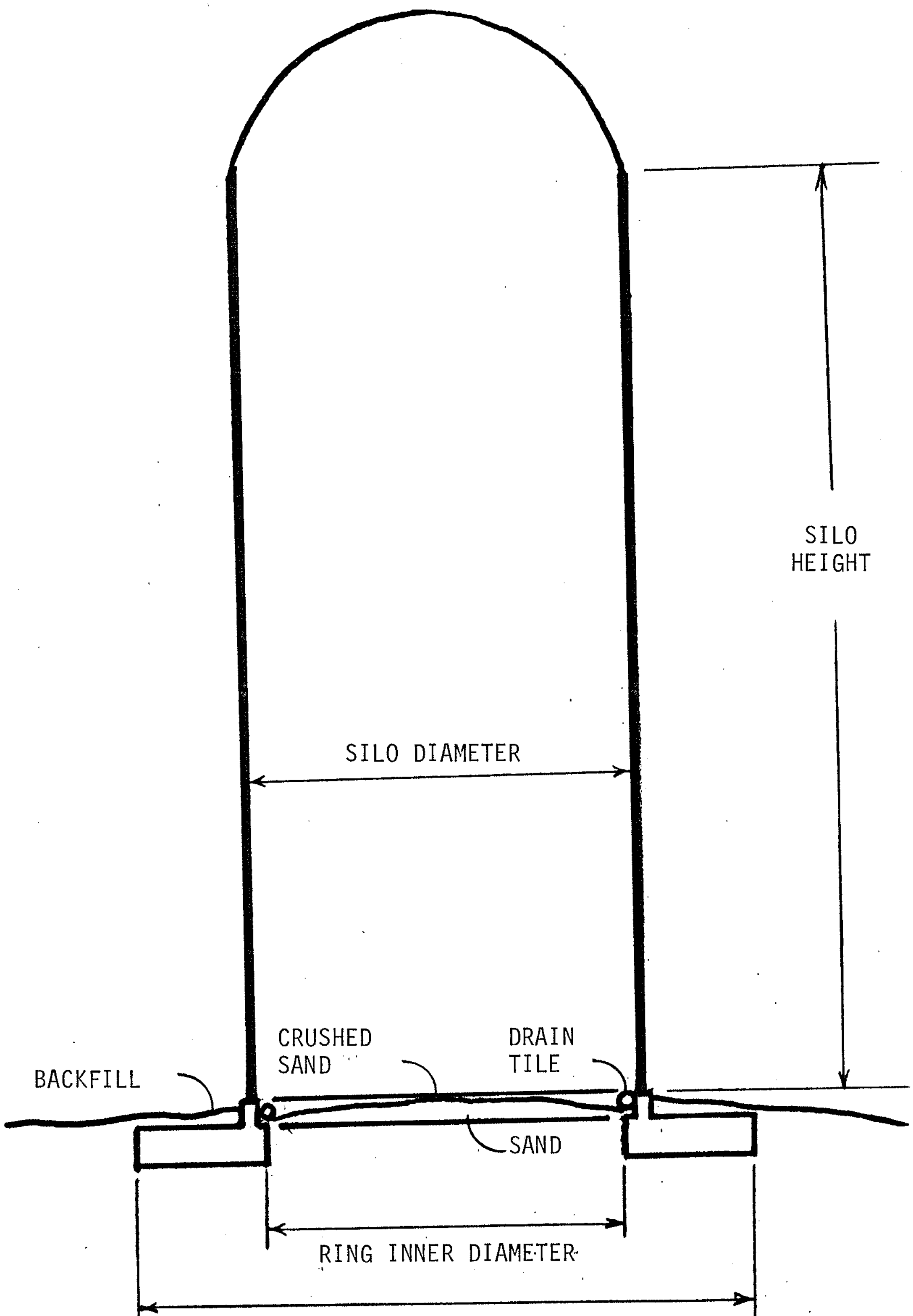


Fig. 3.1: Construction details of silo and foundations.

Soil overburden to the footing depth is due to the mass of material occupying the space inside the ring footing from the floor of the silo to the bottom of the footing. This load can be calculated by the equation:

$$S_o = U \times pi \times R_i^2 \times N \qquad 3.2$$

where S_o = soil overburden load (kN)

U = unit weight of material (kN/m^3)

R_i = inner radius of ring (m)

N = distance, silo floor to footing (m)

pi = 3.1416

Of the total mass of silage in the silo, a portion of it is supported by the walls, due to vertical wall friction, and the rest is supported at the silo floor. The CANADIAN FARM BUILDING CODE (1983) gives an equation for estimating the vertical wall friction load. The equation is:

$$F = (U_s g \times H^2 / 18\ 900) \times (1 - H / 14.2D_s) \qquad 3.3$$

where F = vertical silage friction load per unit circumference (kN/m)

U_s = average mass density of the silage at 70 % moisture content (kg/m^3)

g = acceleration due to gravity = 9.81 m/s^2

H = silage depth (m)

D_s = silo diameter (m)

For a top unloading silo, the floor load is simply equal to the total silage load minus the wall friction load. It should be noted that if the silo is a bottom unloader, the entire silage load should be considered as supported by the walls.

In most silos, 300 mm (Canada Plan Service plan 7411) of the ring extends inside the silo walls and thus carries some of the silage floor load. Because the area of the ring directly loaded by the silage is relatively small, the resultant loading force is added to the wall loading. The resultant force due to silage acting directly on the ring footing is found by:

$$S_r = (gS - F \times 2pi \times R_s) \times (R_s^2 - R_i^2) / (R_s^2) \qquad 3.4$$

where S_r = silage force acting directly on ring (kN)

F = wall load due to friction (kN/m)

S = silage mass (kg)

R_s = radius of silo walls (m)

R_i = inner radius of ring (m)

The vertical wall load due to friction is summed with the structural wall, roof and equipment loads to produce a line load which acts on the ring foundation along the circumference of the wall. This can be stated in the equation form:

$$Q = F + (S_s + S_e + S_r) / (pi \times D_s) \qquad 3.5$$

where Q = Line load on Ring (kN/m)

S_s = load of silo superstructure (kN)

S_e = load of mechanical equipment (kN)

S_r = direct silage force on ring (kN)

D_s = silo diameter (m)

The mass of the ring represents a considerable load and can be accounted for as an uniformly distributed load over the area under the ring. This portion of the contact pressure can be determined by:

$$q_r = U_c \times d \qquad 3.6$$

where q_r = contact pressure due to ring (kPa)

U_c = unit weight of reinforced concrete (kN/m^3)

d = thickness of ring footing (m)

The portion of the silage load supported by the floor, along with the soil overburden load supplies a uniformly distributed load on the area enclosed by the ring. This pressure is evaluated at the footing level in accordance with accepted practice. This component of the loading can be calculated by:

$$q_f = (gS - F \times 2pi \times R_s - S_r) / A_f \qquad 3.7$$

where q_f = distributed load on floor (kPa)

W = silage load (kN)

F = wall friction load (kN/m)

S_r = direct silage load on ring (kN)

A_f = floor area (m^2)

There are, therefore, three load components to consider in order to analysis an entended ring silo foundation. A line load transmitted through the walls, a uniformly distributed load contributed by the ring mass, and a uniformly distributed load inside the ring (See Fig. 3.2).

3.2 Ring Analysis

The silo walls transmit a line load to the ring footing close to the inside diameter of the ring. Typically, the inside of the wall is located 0.200 m to 0.300 m from the inner edge of the ring. The corresponding width of a section of ring is in the neighbourhood of 2.500 m for large silos. This introduces a very large eccentricity. In order to perform a force analysis to find the pressure distribution under the footing, it was decided to model the ring footing as a strip footing.

The assumption that a strip footing is an adequate representation of a ring footing has been made with some reservations. For a strip footing, moment in cross section about the centroidal axis, due to the eccentric load at the wall and the resultant of the contact pressure can be equated. In the case of the ring, the bottom of the ring rotates outward, under load, causing the lower inside area of ring to go into tension, while the upper outside area is in compression (see Fig. 3.3). This action gives the ring shape its inherent stiffness and rigidity. Since a ring does not rotate like a strip under eccentric loading, the effective contact area is greater under a ring footing than a strip footing of equal width. The calculation involved in equating moment at

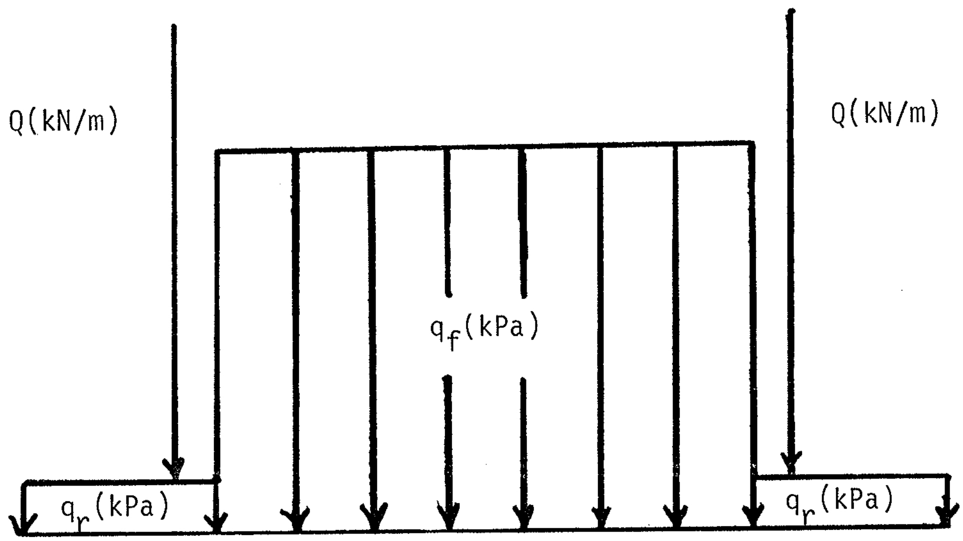


Fig. 3.2: Load components of a tower silo.

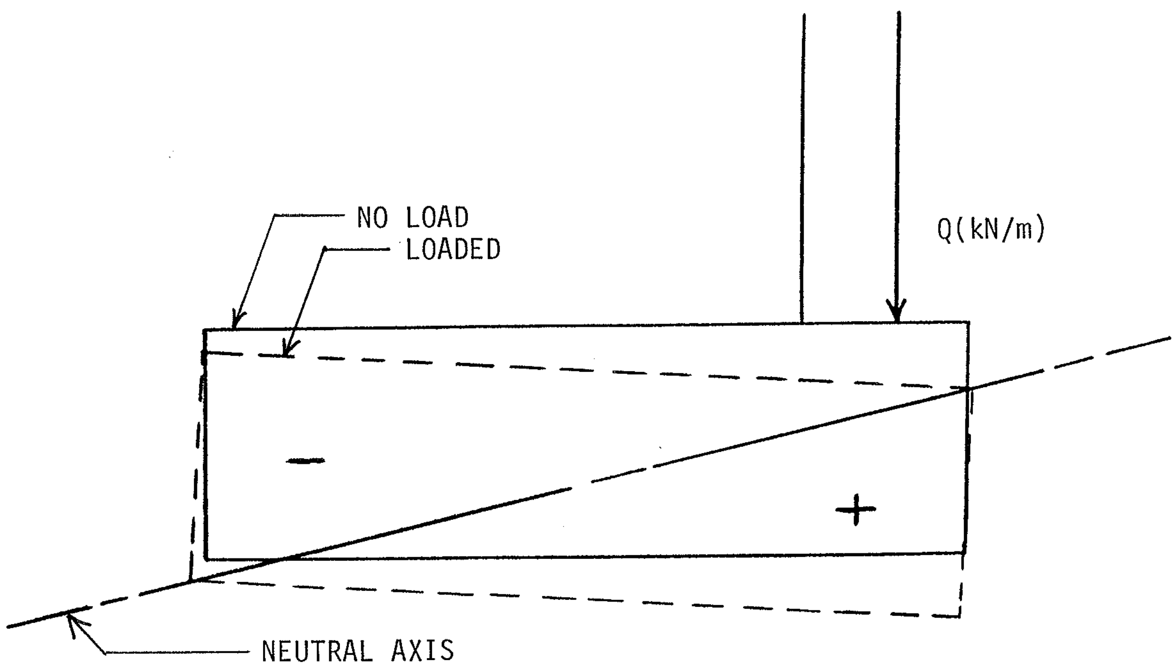


Fig. 3.3: Rotation and settlement of ring footing under eccentric loading.

a cross section of a ring is also severely complicated by the curvature, longitudinal stress, and the fact that the effective bearing area is unknown.

In defence of the decision to model the ring footing as a strip, it is known that the ring effect diminishes as the ratio of the inner diameter to the outer diameter increases. Poulos and Davis (1974) suggest that when the ratio reaches 0.9, a ring acts essentially the same as a strip. The ratio of inside to outside diameter of a ring footing for a typical large silo, 9.100 m by 29.300 m high, on soil with a safe bearing capacity of 240 kPa, is approximately 0.7. Many smaller silos have ratios of 0.5 to 0.6. For all these cases where the ratio of inner to outer diameter is less than 0.9, it has been assumed that the strip action is dominant. By making this assumption, equations which estimate subgrade soil stress due to strip loading may be used. At the early stage of investigation which this study represents, an exhaustive diversion into the nature of the errors which this assumption introduces does not seem justified.

3.3 Equating Forces

By making the assumption that a strip footing approximates the characteristics of a ring footing, the pressure distribution under the ring footing can be determined. Wherever possible, the strip footing model is modified to reflect ring properties. The origins of the horizontal X and vertical Z axes are at the lower left hand corner of a portion of the ring cross section (see Fig. 3.4).

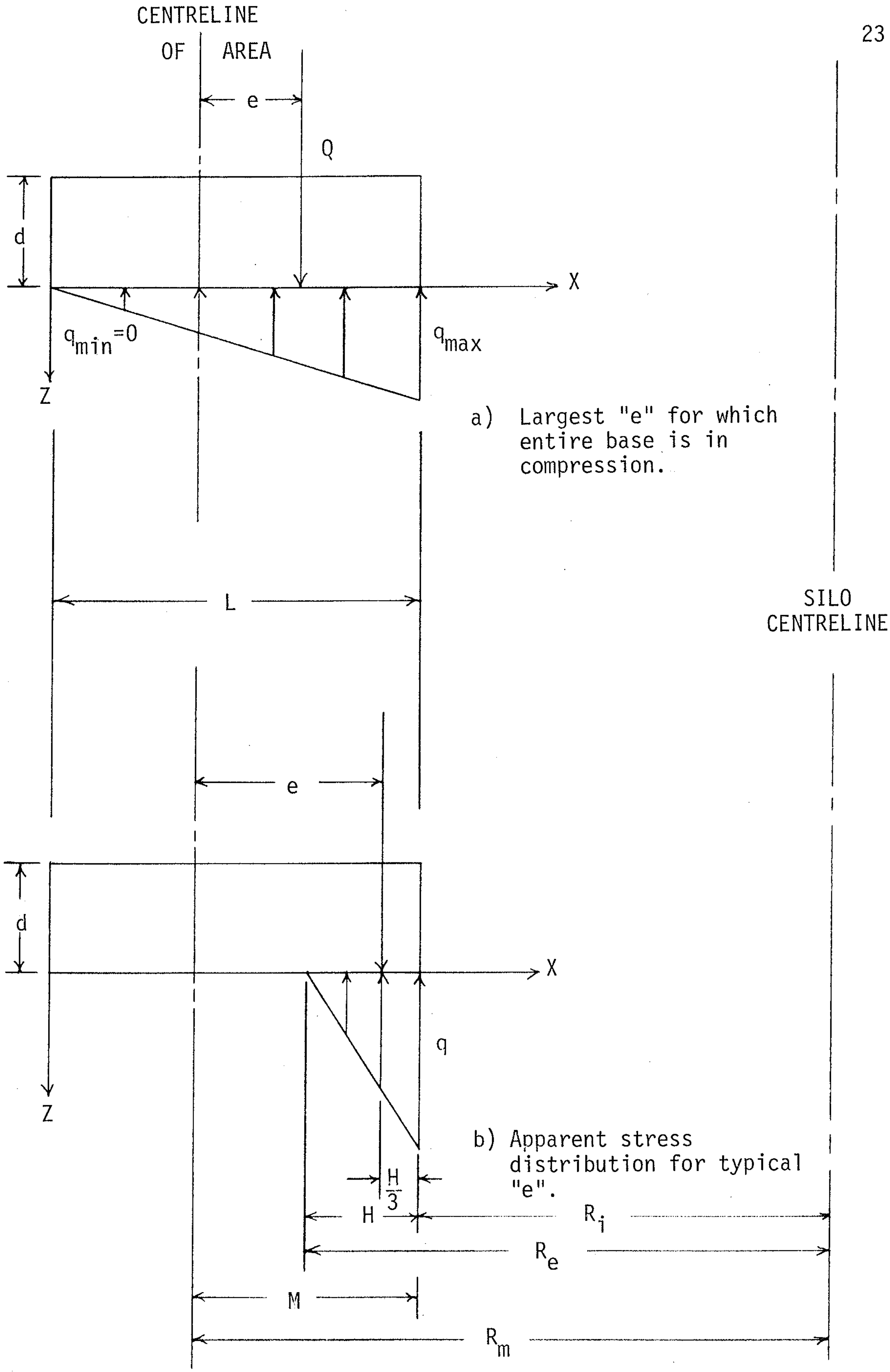


Fig. 3.4: Balancing force and moment on a footing cross-section.

The special case (see Fig. 3.4a) of $q_{min} = 0$, for a rectangular footing of dimensions BxL and load eccentricity 'e' implies that the moment arms of the wall load and the resultant of the contact pressure distribution are of equal length. Summing forces in the vertical direction gives:

$$q_{min} = 0 = Q/BL\ (1 - 6e/L) \qquad 3.8a$$

$$\text{or } e = L/6 \qquad 3.8b$$

Typically, 'e' is greater than L/6 for extended ring footings. This result indicates that only a portion of the footing is being utilized (see Fig. 3.4b).

For equilibrium in the vertical direction:

summing $F_z = 0$

$$Q \times 2pi \times R_s = .5 \times q \times pi\ (R_e^2 - R_i^2) \qquad 3.9$$

where Q = wall load (kN/m)

q = maximum contact pressure intensity (kN/m^2)

R_s = silo radius (m)

R_e = outer effective radius (m)

R_i = inner radius (m)

This reduces to:

$$q = 4QR_s\ /\ (R_e^2 - R_i^2) \qquad 3.10$$

The equation for the radius defining the centre of area of a ring is:

$$R_m = ((R_o^2 + R_i^2)/2)^{1/2} \qquad 3.11$$

R_m defines the equivalent centroid. Eccentricity is measured from the centre of area to the inside surface of the silo wall. The equation for eccentricity is:

$$e = R_m - R_s \qquad 3.12$$

By the laws of statics, the resultant of the triangular pressure distribution under the footing acting at H/3 from the inner edge equals the applied load Q. In order to equate moment about the radius of equal area, the moment arms of the resultant forces acting on the footing must be of equal length. Hence, the relationship;

$$H/3 = M - e \qquad 3.13$$

where $M = R_m - R_i$

Thus, the width of ring being utilized is:

$$H = 3(M - e) \qquad 3.14$$

The outer effective radius can be defined as:

$$R_e = H + R_i \qquad 3.15$$

and the equation for maximum pressure intensity (3.10) becomes:

$$q = 4QR / ((H + R_i)^2 - R_i^2) \qquad 3.16a$$

$$q = 4QR / (H(H + 2R_i)) \qquad 3.16b$$

To assess the preceding development, this method for calculating foundation contact stresses will be applied in an example. The figures are taken from "The Instrumented Tower Silo At Hammond, Ontario" (Bozozuk 1979). The essential statistics of the silo are as follows:

silo radius $R = 3.660$ m

ring outer radius $R_o = 5.945$ m

ring inner radius $R_i = 3.430$ m

load of silage and structure = 9 996 kN

line load at wall, $Q = 267$ kN/m (equation 3.5)

distributed floor load, $q_f = 109$ kPa (equation 3.7)

From equation 3.11, the radius for the centre of area of the ring is:

$$R_m = ((R_o^2 + R_i^2)/2)^{1/2}$$
$$= ((5.945^2 \text{ m} + 3.430^2 \text{ m})/2)^{1/2}$$
$$= 4.853 \text{ m}$$

Equation 3.12 gives the eccentricity as:

$$e = R_m - R$$
$$= 4.853 \text{ m} - 3.660 \text{ m}$$
$$= 1.193 \text{ m}$$

The distance from the inside radius of the ring to the centre of area radius is :

$$M = R_m - R_i$$
$$= 4.858 \text{ m} - 3.430 \text{ m}$$
$$= 1.428 \text{ m}$$

The width of ring utilized, or in compression, from equation 3.13 is:

$$H = 3(M - e)$$
$$= 3(1.428 \text{ m} - 1.193 \text{ m})$$
$$= .705 \text{ m}$$

Equation 3.16 gives us maximum pressure intensity as:

$$q = 4QR_s \;/\; (H(H + 2R_i))$$
$$= 4 \times 267 \text{ kN/m} \times 3.660 \text{ m} \;/\; (.705 \text{ m}(.705 \text{ m} + 2\text{x}3.430 \text{ m}))$$
$$= 733 \text{ kN/m}^2$$

Added to this figure is the pressure due to the weight of the ring footing itself. This is a uniform pressure distributed under the ring and calculated using equation 3.6:

$$q_r = U_c \times d$$
$$= 23.6 \text{ kN/m}^3 \times .76 \text{ m}$$
$$= 18 \text{ kPa}$$

Bozozuk found the allowable bearing capacity of the soil at this site to be 351 kPa based upon in situ vane shear tests. Based on the preceding calculatiions, the foundation has a safety factor of:

$$351 \,/\, (733 + 18) = .47$$

It is unreasonable to accept this figure as representative of the entire foundation because it implies that only 47% of the load can be supported without a bearing capacity failure. The actual foundation is, however, performing reasonably well. The possibility of a bearing capacity failure is not an issue.

The very low (0.47) safety factor does support the statement made by Bozozuk (1974) that local overstressing can occur due to eccentric wall position on the ring. The actual silo at Hammond also displayed differential ring settlement of up to 32 mm which supports Bozozuk's statement.

If the conditions causing a safety factor of considerably less than 1.0 occur under the inner edge of the ring, they exist for only a short time. The soil directly under the ring will deform and redistribute stress until local yielding stopped, or the entire ring area was acting in compression.

Information collected by Bozozuk using Gloetz earth pressure cells at the Hammond site confirmed that a non-uniform contact pressure existed across the width of the ring footing. At one location on the ring, pressure measured under the heel was twice that measured under the toe. The degree of pressure non-uniformity was consistently high during the first year of operation, but reduced as time went on. Because the danger of bearing capacity failure is highest at the early stages of consolidation due to excess pore water pressure, and further experimental data is unavailable, a conservative design approach was adopted. A 2:1 heel to toe pressure distribution will be assumed. Also, traditional engineering practice requires that design be for the most severe expected loading condition.

To evaluate the design assumption that the pressure distribution across the width of the ring footing forms a 2:1, heel to toe ratio, again use the Hammond silo (Bozozuk 1979 b), and balance vertical forces (see Fig. 3.5).

Taking the pressure induced by the footing weight into account, and calculating the contact area of the ring A_r by:

$$
\begin{aligned}
A_r &= \text{pi}\ (R_o^2 - R_i^2) \\
&= \text{pi}\ (5.945^2\ \text{m} - 3.430^2\ \text{m}) \\
&= 74.1\ \text{m}^2
\end{aligned}
$$

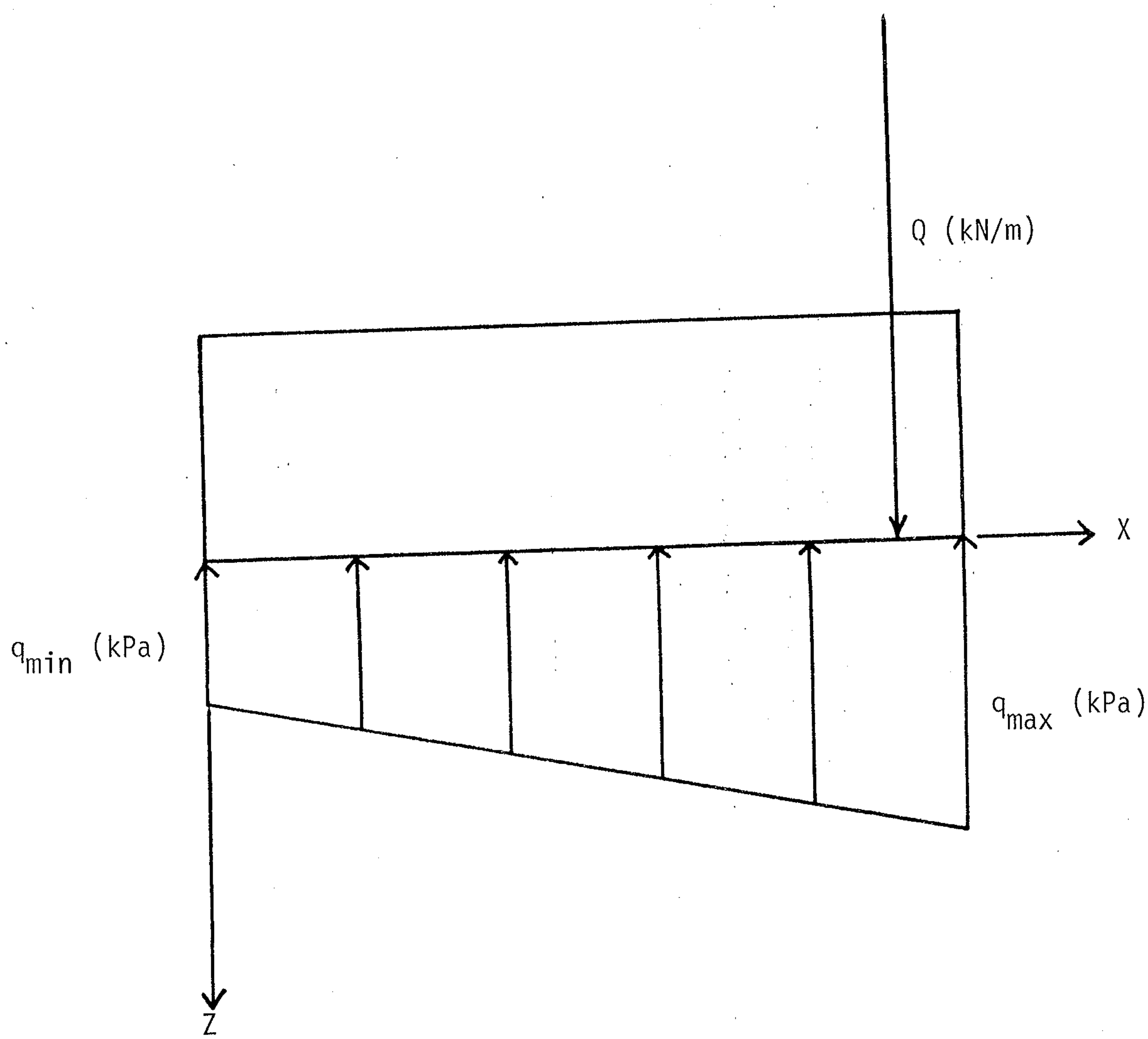


Fig. 3.5: 2:1, heel to toe, contact stress distribution on a section of ring footing.

then the weight of the ring plus the load transmitted through the walls equals the resultant force of the contact pressure. In equation form:

$$q_r \times A_r + Q \times 2pi \times R = 1.5 \times q \times A_r \qquad 3.17$$

rearranging

$$q = (q_r \times A_r + Q \times 2pi \times R) / (1.5 \times A_r)$$

$$= 18 \text{ kN/m}^2 \times 74.1 \text{ m}^2 + 267 \text{ kN/m} \times 2pi \times 3.66 \text{ m} / (1.5 \times 74.1 \text{ m}^2)$$

$$= 67 \text{ kPa}$$

The safety factor under the footing now becomes:

$$351 \text{ kPa} / (2 \times 67 \text{ kPa}) = 2.62$$

This result is in accordance with observed foundation performance and the average pressure of;

$$1.5 \times 67 \text{ kPa} = 100 \text{ kPa}$$

is close to the measured average contact pressure of 84 kPa. This encouraging result lends support to the assumption of a 2:1 pressure ratio across the width of the footing.

The contact pressure under the floor at the footing level is normally lower than the pressure under the footing. For the Hammond, Ontario example the contact pressure was measured at 121 kPa under the centre of the floor. The earth pressure cell was placed under 540 mm of crushed rock and gravel. The calculation for contact pressure at this depth is:

$$q_d = q_f + .54U \qquad 3.18$$

where q_d = contact pressure (kPa)

q_f = silage load = 109 (kPa)

U = unit weight of gravel = (16 kN/m^3) (Bowles 1982)

thus q_d = 109 kPa + .54 m x 16 kN/m^3

= 118 kPa

Hence, the calculated (118 kPa) and measured (121 kPa) contact pressures under the silo floor are quite close.

For design purposes, the calculation should be made at the footing depth of 1.3 m as is standard engineering practice.

$$q_d = 109 \text{ kPa} + 1.3 \text{ m} \times 16 \text{ kN/m}^3$$

$$= 130 \text{ kPa}$$

For the tower silo at Hammond, Ontario the actual safety factor against bearing capacity failure under the floor area is:

$$\text{S.F.} = 351 \text{ kPa} / 130 \text{ kPa}$$

$$= 2.70$$

Using the simplifying assumption that the entire load of the structure is uniformly distributed across the base of the structure, the safety factor becomes misleadingly high. For the Hammond, Ontario example, the total load was 9 996 kN plus the soil overburden of 769 kN (equation 3.2). The contact pressure was:

$$P = (9\ 996 + 769)\text{kN} \ / \ \text{pi} \times 5.945^2 \text{ m}$$
$$= 97 \text{ kPa}$$

The safety factor as determined from Bozozuk is;

$$SF = 351 \text{ kPa} \ / \ 97 \text{ kPa}$$
$$= 3.62$$

The overestimation of the safety factor with respect to the previously presented analysis is:

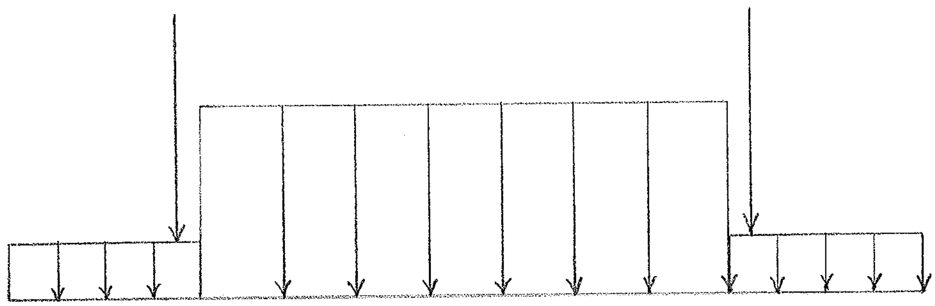
$$(1 - 2.70 \ / \ 3.62) \times 100 = 25.4\%$$

For the particular example of the Hammond, Ontario tower silo, the foundation does supply an adequate safety factor against bearing capacity failure. However, an overestimation by approximately 25% of the safety factor has the potential to lead to serious stability problems in cases where soils investigations are not representative of the actual conditions, or when highly variable conditions exist.

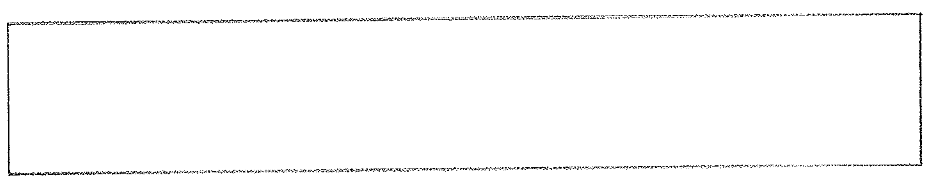
Figure 3.6 shows the proposed composite loading scheme and the conventional loading scheme. Clearly the composite footing model more accurately reflects the actual load distribution.

The preceding analysis suggests a load transfer mechanism for the extended ring foundation which is consistent with experimental observations and engineering principles. Due to the large load eccentricity on the ring footing, localized yielding occurs in the supporting soil under the heel of the ring. This results in a 'dishing out' of the ring. That is, differential settlement between the inner and outer edges of the ring. This distortion causes stress redistribution in the direction of a more stable equilibrium. Because settlement is stress induced, a footing will 'bed in' in a manner which tends to equalize contact pressure. However, the majority of settlement is time dependent. This means for a significant length of time, a distinct non-uniformity of contact pressure exists under the extended ring footing. A contact pressure ratio of 2:1 has been chosen based on available experimental data.

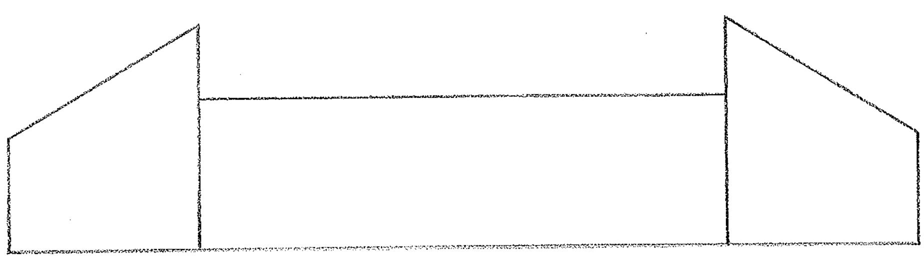
With time, pressure non-uniformity across the base of the silo will disappear. This stress redistribution is a result of expulsion of pore water associated with primary consolidation and has been observed in the field (Bozozuk 1979 b). Normally contact pressure uniformity will not occur until after repeated loading cycles and consolidation is well advanced. Since the critical period for bearing capacity is during the



i: Silo loads.



ii: Round footing load distribution.



iii: Composite footing load distribution.

Fig. 3.6: Comparison of contact stress distribution for Round and Composite footing models.

early stages of consolidation, it is proposed that a 2:1, heel to toe, contact pressure distribution be assumed for extended ring foundations.

Chapter IV

COMPUTER MODEL

4.1 Introduction

The location, magnitude and type of surface load determine the location and magnitude of the stresses in the foundation soil. Presently, the load from a tower silo is modeled as being uniformly distributed over the whole area bounded by the ring footing. As shown in the preceding section, this can lead to significantly underestimated contact pressures. The alternative is to treat the load carried by the ring and the load carried by the floor separately and combine their contributions to obtain the subsurface stress distribution. To accomplish this, a mathematical model was developed.

The extended ring foundation was considered to be comprised of two distinct parts, hence the name composite footing. The two parts are the ring footing itself, and the floor area, or the area inside the ring. This concept was used to develop a computer program which solves for stress under an extended ring silo foundation. The program was written in BASIC programming language and run on a Corona PC-21 personal computer. The purpose of the program was to calculate the loads for a given silo and to find pressure isobars in terms of a percentage of a reference surface load. These pressure bulbs could then be plotted for observation and comparison purposes.

The following sections first detail the theory involved in estimating stress increase in the subgrade due to surface loading.

Secondly, details of how the program is structured and controlled will be outlined. Finally, the actual computational methods will be presented along with the methods employed for storage and retrieval of the resulting information.

4.2 Stresses due to Footing Pressures

As previously described, it was assumed that the ring footing carries a 2:1 contact pressure distribution, heel to toe, across the width of the ring. This conclusion was reached on the basis of its compatibility with available experimental results as explained in Chapter III. By modeling the ring as a strip area carrying a uniform pressure of magnitude W plus a superimposed linearly-increasing pressure also of magnitude W (maximum), stress at any point under the ring can be determined. By defining angles "a" and "b" in figure 4.1 as:

$$b = \mathrm{Tan}^{-1}(X-H/Z) \qquad 4.1\ a$$

$$a = \mathrm{Tan}^{-1}(X/Z) - b \qquad 4.1\ b$$

(Craig 1978)

the equation for stress due to a uniformly loaded strip area becomes:

$$S_z = (W/pi)(a + \sin a \cos(a + 2b)) \qquad 4.2$$

and the equation for stress due to a linearly increasing load on a strip area becomes:

$$S_z = (W/pi)(X/H\ a - 1/2 \sin 2b) \qquad 4.3$$

(Craig 1978)

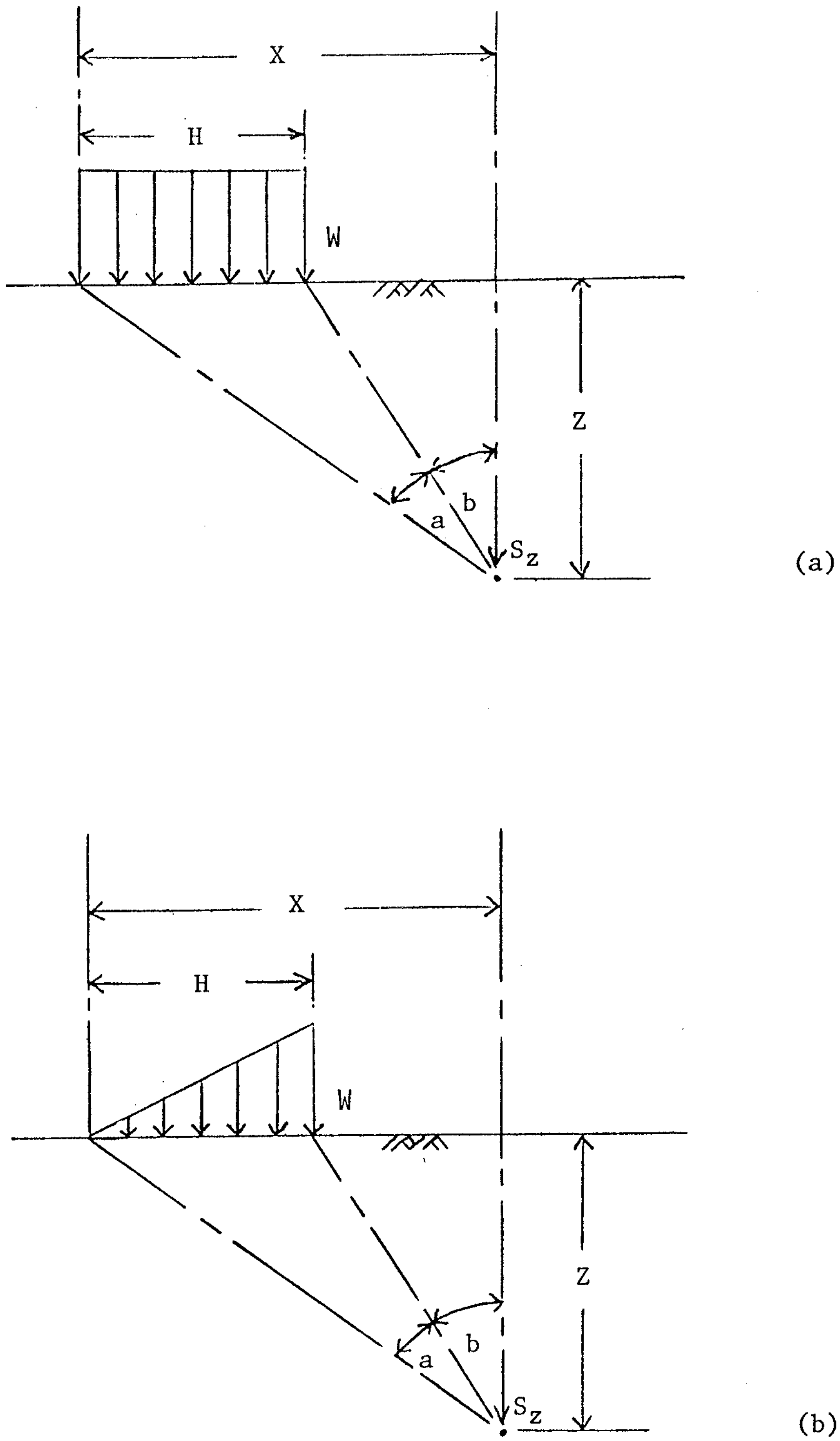


Fig. 4.1: Stress due to (a) uniform pressure,
(b) linearly increasing pressure, on a strip area.

These equations are based on elastic theory and assume homogeneity and isotropy in a semi-infinite domain. Stress increase in the subgrade is dependent on the size of the loaded area, direction of the radius vector (X distance), and the depth (Z distance) of the point of interest.

The Boussinesq method for evaluating soil pressure, models a point load on the surface of an infinitely large homogeneous, isotropic, weightless, elastic half-space (see Fig. 4.2). The equation is in the form of:

$$q_v = 1.5Q \cos^5\theta/(pi \times Z^2) \qquad 4.4$$

where q_v is the stress intensity due to the point load Q. The stress increase is axially symetric. By setting:

$$\tan \theta = r/Z \quad \& \quad R^2 = r^2 + Z^2$$

then, $$\cos^5\theta = (Z/R)^5$$

hence, $$q_v = 1.5Q \times Z^3 / (pi \times R^5) \qquad 4.5$$

To obtain stresses due to distributed loads, a numerical integration technique is used. This involves modeling a uniformly distributed load as a series of point loads acting on equal areas. Bowles (1982) suggests treating a round footing as an equivalent square footing by setting length equal to the square root of the area of the round footing. It was decided not to use an equivalent square area, but to

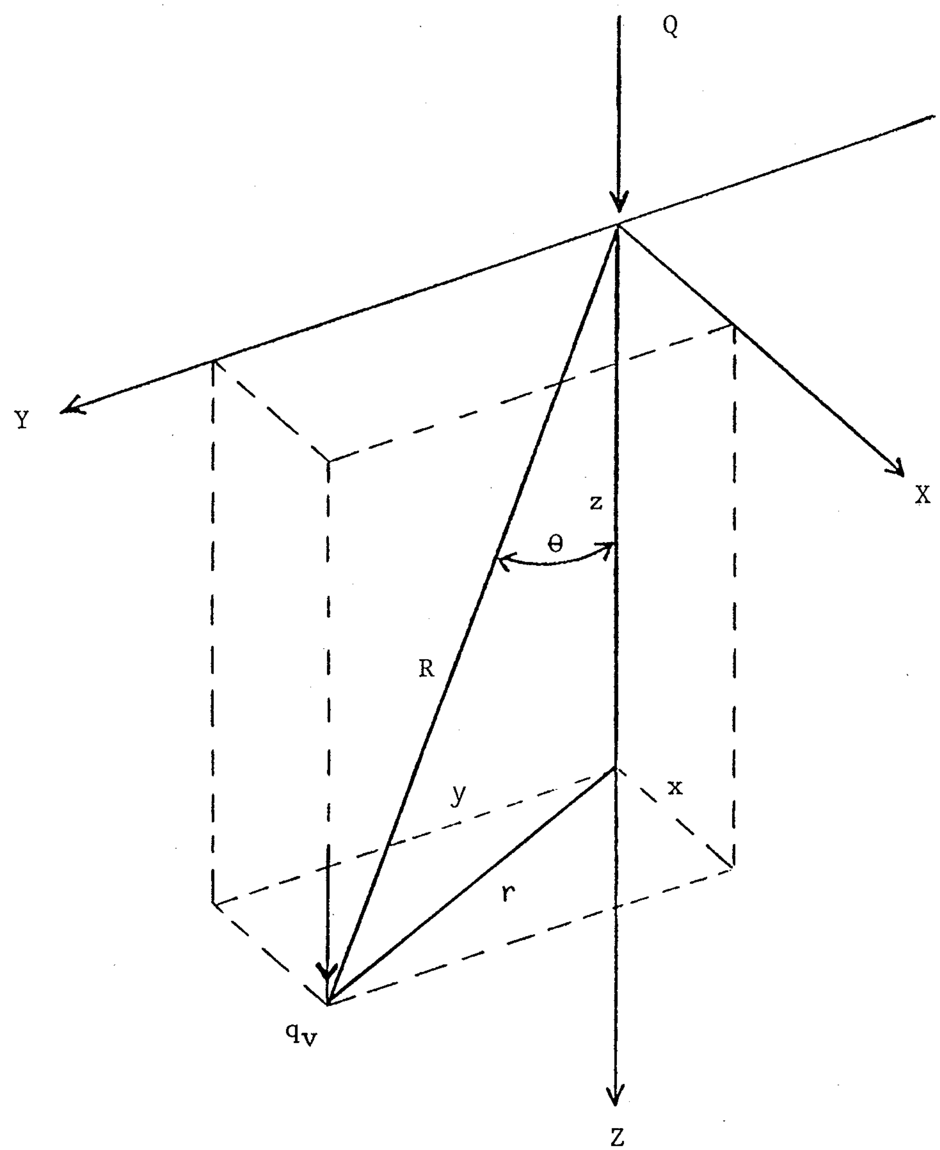


Fig. 4.2: Stress increase due to point load based on Boussinesq.

subdivide the round area. The limited simplification afforded by the equivalent square area did not appear justified in view of an expected compromise in accuracy. Also, it would be impossible to investigate the interaction of the floor area and ring footing if the floor was modelled as a square area because the geometry would be incompatible. Bowles (1982) also suggests that subdividing a footing into areas smaller than 0.100 m^2 does not significantly improve the accuracy. Consideration of the size range of tower silos and computer computation time dictated the number of subdivisions. The round footing is divided into 20, 18 degree pie-shaped sectors. Each sector is further subdivided into 10 equal areas (see Fig. 4.3). This creates 200 equivalent areas which each carry an equal point load to simulate a uniformly distributed load. For example, the element area of a 6.500 m circular footing is .166 m^2. The stress at any point can now be computed as a summation of the stresses contributed by each individual point load.

4.3 Computer Program

Pressure bulb or stress isobar diagrams are a common way of illustrating the zone of significant stressing. A pressure bulb is a bulb-shaped line joining points of equal stress increase. Newmark's Chart has traditionally been used to produce these diagrams. It estimates stress increase under a footing of any shape by graphical methods. Developing stress isobars by this method is extremely tedious and time consuming. The advent of the digital computer provided numerical methods for solving for stress increase due to surface loading.

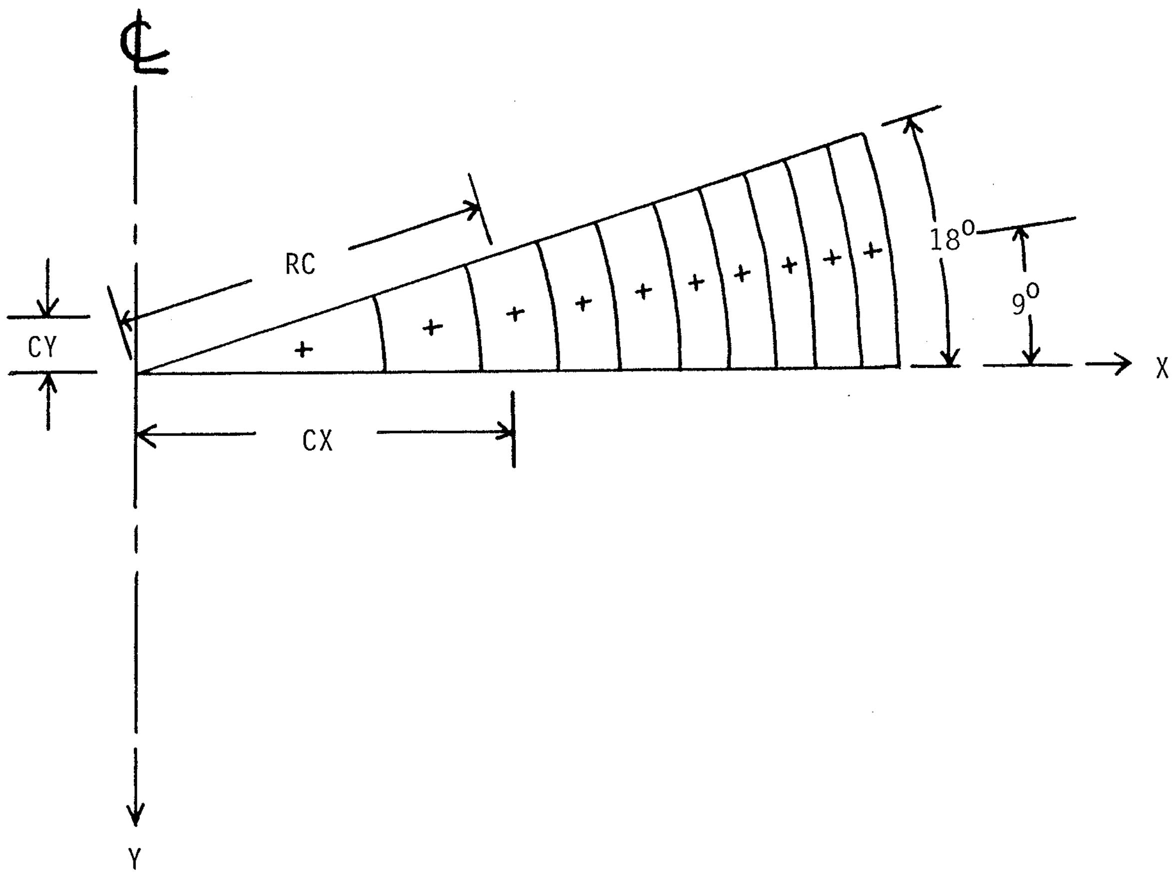


Fig. 4.3: Distribution of point loads.

To meet the objectives of this study, pressure bulb diagrams for various load conditions of concrete tower silos were required. Two loading formats were to be considered. The first is a "composite" loading format where contact pressures under the ring and under the floor are calculated separately. The second is a "round" loading format where contact pressure is distributed uniformly over the entire silo base. Numerical models were developed for both these load conditions so that stress increase due to either loading format could be determined.

4.3.1 Input & Load

The program allows choice of either the 'ROUND' or the 'COMPOSITE' footing model. The 'ROUND' footing model assumes the entire weight of the silo is uniformly distributed over the entire base area. The 'COMPOSITE' footing model assumes the load is proportioned between the extended ring footing and the silo floor.

The program user is then prompted for the critical dimensions of the silo, its foundations, and the depth of silage. Control is then passed to a subroutine which calculates loading based on the chosen footing model. Load values are then displayed to the user. The subroutine uses the following values and source for load estimation:

Unit Weight, reinforced concrete; 23.56 kN/m^3

Unit Weight, overburden soil; 16.00 kN/m^3

Density of silage; Canadian Farm Building Code (1983)

'Reference pressure' is the term used to denote the applied bearing pressure to be used to calculate the safety factor against bearing capacity failure. For the round footing model, 'reference pressure' is the pressure under the silo base. For the composite footing model, 'reference pressure' is the larger of the maximum pressure under the ring footing or the pressure under the silo floor. If contact pressure is calculated to be higher under the ring, it is likely the size of the affected area will be limited since the pressure distribution is linearly increasing, from the outer to the inner edge of the ring. However, to avoid the danger of excess settlement under the heel of the footing, stress at this point is used to determine the safety factor. Bozozuk (1974) recognized the danger of local overstressing under the heel of the ring footing. He noted that the ring foundation "may tip or twist, deform and crack into individual sections, and destroy the monolithic performance of the unit". For these reasons the maximum contact pressure under the heel of the ring footing is used as the 'reference pressure'.

4.3.2 Axes Definition

To produce pressure bulbs, stress increase at every point in an appropriate grid is evaluated. Coordinates of the points which pass acceptance criterion are saved on a floppy disk. The saved points, when plotted, form curves which represent the stress isobars. The location of the axes on a typical foundation is shown in Fig. 4.4 . The orientation of the search grid set up in the computer program is shown in Fig.4.5.

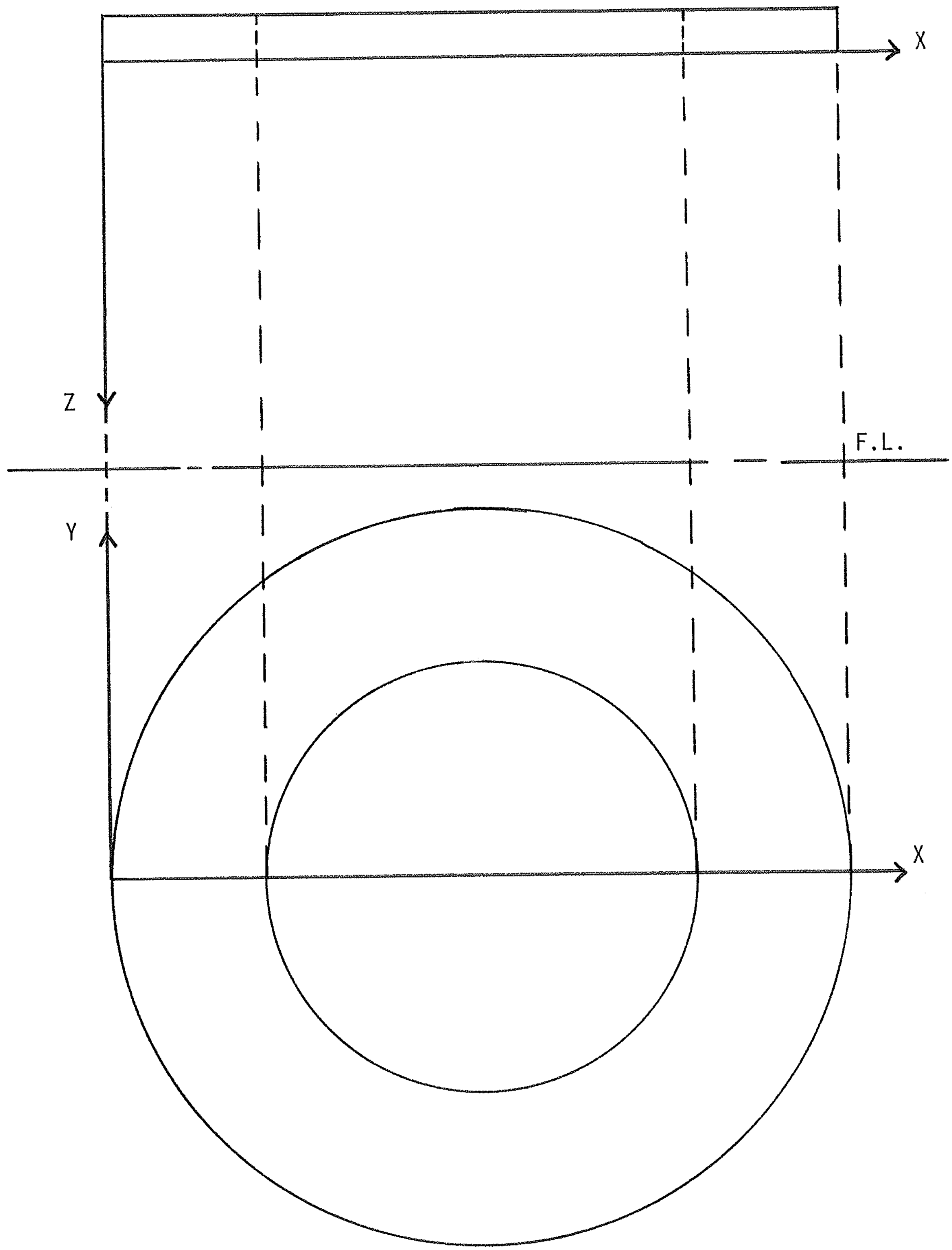


Fig. 4.4: Location of axes, elevation and plan views.

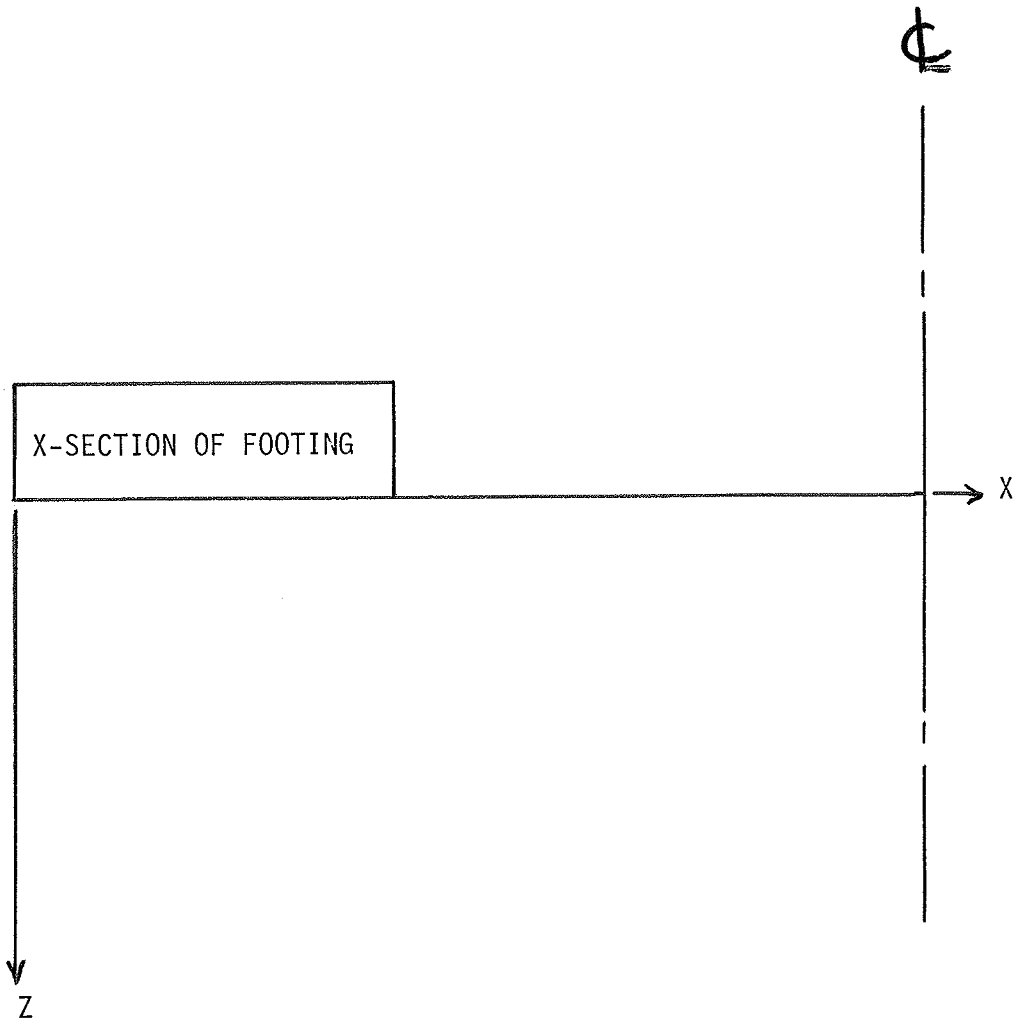


Fig. 4.5: Location of grid area, evaluated for stress increase in the process of solving for pressure bulbs.

The origin of the grid is located at the outer edge of the ring to conform with equations 4.1 and 4.2 for stress increase due to a loaded strip area (see Fig. 4.1). The X coordinate is initialized with a negative value to ensure that significant stressing outside the foundation boundary is detected. It extends only to the silo centreline because of symmetry of loading. The Z coordinate extends to 1.2 times the outside diameter of the ring. These limits have been found to encompass the pressure bulbs representing stress increases greater than 20% of the reference applied load.

4.3.3 Increment Size and Execution Time

The program will prompt the user to input increment size for X and Z grid coordinates. This determines the number, and hence, the spacing of the locations to be evaluated for stress increase. Increasing increment size has the effect of reducing execution time but results in less clearly defined pressure bulbs. Decreasing increment size improves accuracy but increases execution time. After specifying increment size, the program estimates execution time and asks the user if this is acceptable. Execution time is calculated on the basis of 24.5 s to evaluate each grid point. If execution time is unacceptable, new increment sizes can be declared.

4.3.4 Computational Techniques

Two important functions of the computer program are to define the locations which are to be tested for stress increase and define the locations of the loads. This requires numerically modeling both the X-Z plane which forms the search grid and the X-Y plane which contains the foundation in plan. Every point on the search grid is evaluated for stress increase and the coordinates of the chosen points define the pressure bulbs. In the X-Y plane, every point load used to approximate the uniform floor loading must also be located so that its effect at any point in the search grid can be evaluated.

In the computer program, each point (X, Z) which is evaluated for stress increase, includes a contribution from the load carried by the ring foundation and from each of the 200 point loads used to approximate the circular distributed load. To satisfy the Boussinesq equation, each point load must be defined by its X - Y coordinates.

Step one in this process is to calculate the 10 radii at which these point loads, representing the uniformly distributed floor load, are located. This is done in a subroutine (see Fig. 4.6). By visualizing the circle as 10 concentric rings of equal area, it is clear that the widths of the rings will decrease as distance from the centre increases. Since the rings are all equal in area, the inner and outer, and hence, the centroidal radius, can be found. The point loads are located on the centroidal radii (see Fig. 4.3). As each radius is calculated in the subroutine, it is stored in an array for later use.

A nested loop is used next to solve for the X - Y coordinates of the point load and also to evaluate their influence. The outer loop defines the angle at which each point load occurs, and the inner loop is used

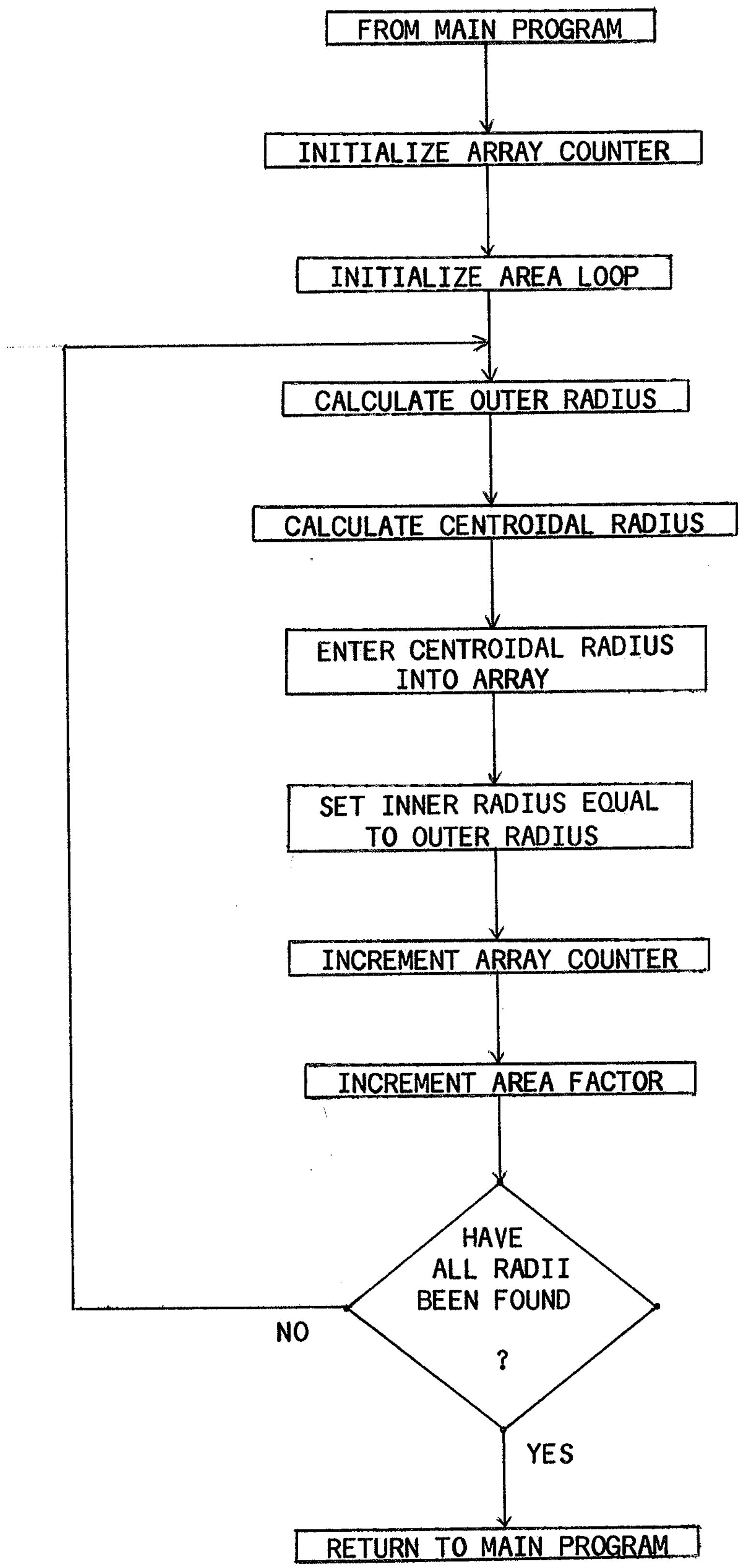


Fig. 4.6: Flowchart of subroutine to calculate radius of each point load.

to recall the appropriate radius from the array (see Fig. 4.7). By knowing the radius and angle of each point load the X - Y coordinates can be solved by simple trignometry. To solve the Boussinesq equation for stress increase due to a surface point load, the distance (R) from the load to the point of interest must be calculated (see Fig. 4.2). This is done with the equation:

$$R = (x^2 + y^2 + z^2)^{1/2} \qquad 4.6$$

where x = distance in X direction from point load to point being evaluated

y = Y coordinate of point load

z = depth of point being evaluated

It should be noted that the X - Y coordinates are measured from the centre of the circle, that is the centre of the silo. To obtain the X direction distance from a point load to the location being evaluated a coordinate shift must be performed. The magnitude of the shift is the difference of the X coordinate of the centre of the silo, subtracted by the X coordinate of the point being evaluated for stress increase. By adding this amount to the X coordinate of point load, the correct X distance is obtained.

Inside the nested loops just described, the influence of each of the 200 point loads are calculated and a running sum maintained. This is summed with the contribution from the load borne by the ring footing to give the resultant stress increase.

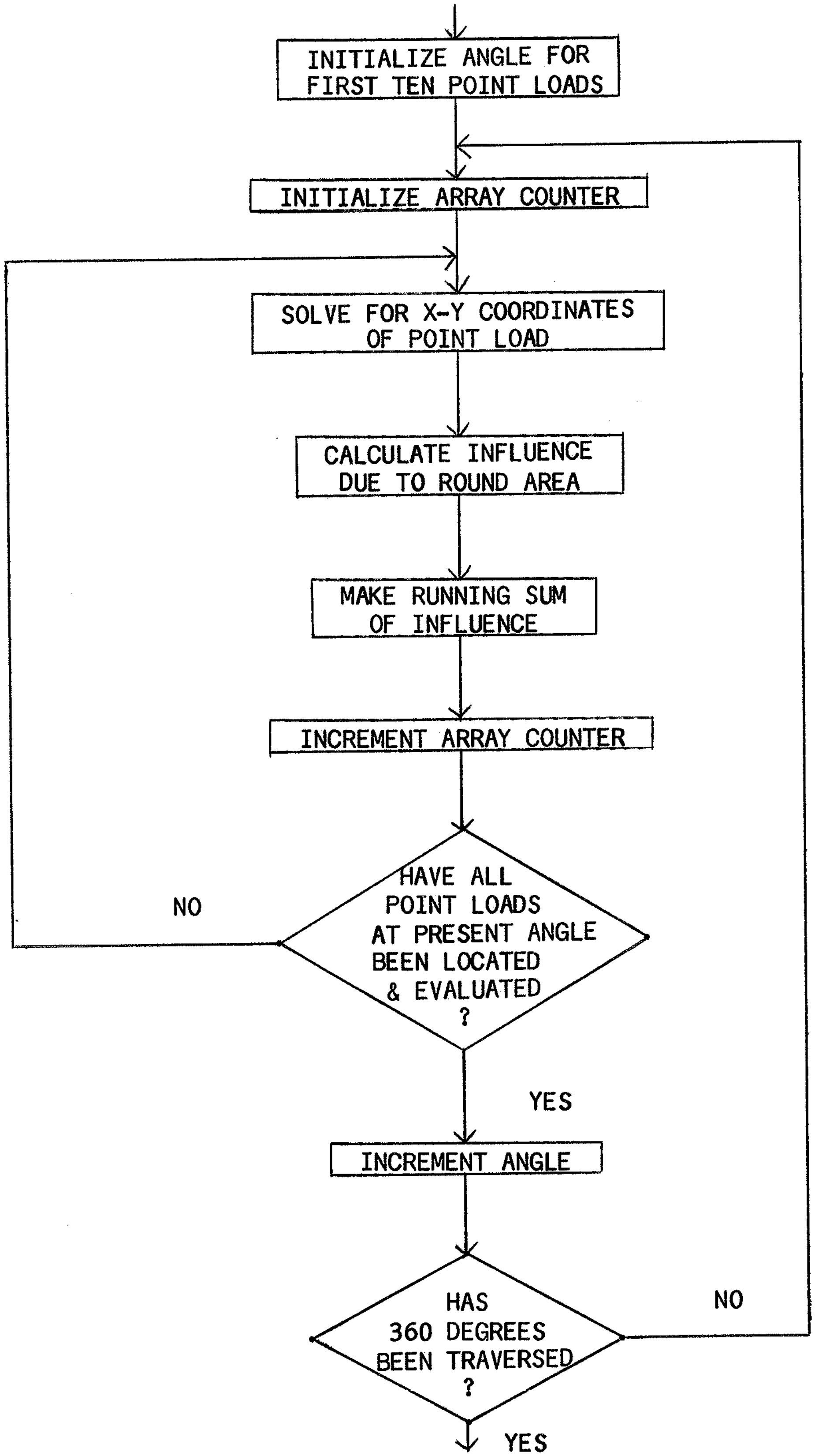


Fig. 4.7: Flowchart of routine to calculate influence from an uniformly loaded round area.

4.3.5 Acceptance Criterion

The most common method of presenting stress isobars is as a percentage of the contact pressure. To this end the computer program accepts for storage the coordinates of the points which have stress increase values close to 20%, 40%, 60%, 70%, 80% and 90% of the reference pressure. Actual stress increase at each grid point is evaluated by multiplying the influence from the ring and floor by their respective load magnitude. This is done because the internal calculations use unit loads to reduce program execution time. This stress value is then divided by the reference contact pressure to get a decimal fraction figure.

For a location on the grid to be accepted as a point on a pressure isobar it must pass a two stage acceptance criterion. First, the ratio of stress increase to reference pressure must be close to one of the aforesaid values. The ratio is checked to see if it fits within a narrow range around each specified value. The ranges were fixed after many trials using various values. The goal was to find the range which was small enough to give well defined pressure bulbs without having the points cluster, and large enough so that gaps would not be left in the isobars. Secondly, the coordinates cannot be too close to the most recently accepted point for that isobar. If it is close, it is rejected.

4.3.6 Storage and Retrieval

Once a location on the grid is found which represents a point on a particular pressure isobar, it is stored in the appropriate array. There are six arrays, one for each isobar. The X and Z coordinates of each point are stored as well as the ratio of stress increase to reference contact pressure. This allows the user to discern accuracy of fit. Enough space is provided in the arrays to handle 50 points for the 20%, 40% and 60% isobars and 30 points for the 70%, 80% and 90% isobars.

Storage of the accepted grid points is achieved by reading the contents of each array into its own sequential data file. Each array is read into the data file after all points on the grid have been evaluated. This leaves a record of the coordinates for each pressure bulb on disk for later use. A BASIC program to access these data files can be found in APPENDIX C.

Chapter V

RESULTS

5.1 Contact Stresses

To compare the current practice of assuming the total weight of a silo and contents are uniformly distributed over the entire area of the foundation base with composite footing theory, both methods were applied to a number of silos for which sufficient documentation was available. Information required for the comparison was complete structural dimensions and bearing capacity of the soil. The author's computer program load calculation routine was used to determine the reference contact pressure for both round and composite footing theory. By dividing the bearing capacity by reference contact pressure, the associated safety factor was found. The resulting safety factors predicted by the two loading theories formed the basis for comparison.

Two of the 30 examples used in this study are completely described in the literature. They are the Hammond silo (Bozozuk 1979 b) and the Richmond silo (Bozozuk 1979 b). The other 28 are from a performance survey of silos in Quebec (Morin and Bozozuk 1983). The information in this latter report did not include the interior diameter of the ring footing, the thickness of the silo walls, or the bearing capacity of the soil. To make use of this source, several assumptions were made. Based on information from Turnbull et al. (1979) and the Canada Plan Service plan 7411 it was assumed that the distance from the inner surface of the silo wall to the inner edge of the ring was 0.300 m. Cast-in-place type silos were assigned a wall thickness of 0.150 m and concrete stave silos a thickness of 0.063 m. These figures were also taken from Turnbull et al. (1979). The problem of solving for bearing capacity was eliminated by calculating the expected reference load under round footing theory and multiplying by the safety factor reported in the Morin and Bozozuk (1983) study. It was further assumed that the reported safety factor was based on a depth of silage equal to the height of the silo walls. It is recognized that bearing capacity derived in this way could result in inaccurate estimates. Unfortunately, the actual figures used by Morin and Bozozuk are not available. The derived bearing capacity values are only used to compare safety factors resulting from round and composite footing theory. It should be noted that the safety factors produced by the composite footing model may not accurately reflect the conditions at that particular location however they are valid for comparison purposes.

Tables 5.1 and 5.2 are reprints from Morin and Bozozuk (1983). These tables contain descriptions of the 28 silos and their performance information. Table 5.3 contains this author's values for bearing capacity reference contact pressure, and safety factors for the 28 sites plus the two previously mentioned by Bozozuk (1979 a and b). Data for Table 5.3 was generated with the author's computer program. By inputting the dimensions of each silo, the loading characteristics of both round and composite footing theory were calculated. Reference contact stress is reported for both ring and floor areas. Safety factor is calculated by dividing the soil bearing capacity by contact

Table 5.1: Reprinted from Morin and Bozozuk 1983 Performance of concrete tower silos on clays in Quebec. Canadian Agricultural Engineering, Vol. 25, No. 1, p. 81-88.

TABLE I. DESCRIPTION, STABILITY AND PERFORMANCE OF CONCRETE TOWER SILOS

Location no. (see Fig. 1)†	Silo dimensions (diam. × ht.) (m)	Type‡	Year built	Foundation: Dimensions (thickness × outside diam.) (m)	Foundation: Type§	Average settlement (4 cycles) (mm)	Tilt (% height)	Factor of safety	Perf. (Table II)
1	5.5×18.3	CS	1975	0.6× 8.2	ADN	155	0.27	2.16	C
2[1]	6.1×18.3	CP	1976	0.6× 8.8	BSFDE	27	0.05	2.62	A
3	6.1×18.3	CS	1975	0.6× 9.4	ADN	27	0.38	3.61	B
4	6.1×18.3	CP	1975	0.6× 7.6	BSFDE	67	0.05	2.50	B
5	6.1×18.3	CP	1975	0.6×10.7	BSFDN	408	0.37	1.68	D
6	5.5×14.6	CP	1975	0.9× 9.1	HSFDN	70	0.47	2.36	B
7	4.9×17.1	CP	1975	0.5× 7.9	BSFDN	52	0.35	3.04	B
8	5.5×15.8	CP	1975	0.6× 9.4	BSFDE	259	1.16	1.80	D
9[2]	5.5×18.3	CP	1974	1.1× 7.3	BSFDE	116	0.21	2.31	C
10	5.5×17.1	CP	1975	0.6× 8.5	BSDCE	91	0.26	3.32	C
11	5.5×15.8	CP	1975	0.6× 9.4	BSFDE	143	0.84	3.52	C
12[3]	4.9×17.1	CP	1975	0.6× 8.5	BSFDE	174	1.12	1.80	D
13	6.1×17.1	CP	1975	0.9× 9.4	BSFDN	351	2.42	1.73	E
14	6.1×18.3	CP	1975	0.8× 8.5	BSFDE	195	0.14	2.42	D
15	6.1×21.9	CP	1975	0.6× 9.0	BSFDE	55	0.62	2.29	B
16	6.1×16.8	CS	1975	0.6× 7.9	ADN	3	0	4.03	A
17[4]	6.1×18.3	CP	1974	0.6× 9.1	BSFDE	300	1.20	2.59	D
18[5]	6.1×18.3	CP	1976	0.6×12.2	BSFDN	442	0.43	1.61	E
19	4.9×18.3	CP	1975	0.6× 7.6	BSFDN	219	0.41	1.73	D
20	4.9×19.5	CP	1975	0.8× 6.4	BSFDE	24	0.29	3.28	B
21[6]	6.1×15.2	CS	1975	0.6× 8.2	BDN	402	1.77	1.84	E
22	5.5×16.8	CS	1975	0.6× 8.1	ADN	21	0.05	5.22	A
23	6.1×15.2	CS	1975	0.6× 9.1	ADCN	61	0.20	3.52	B
24	5.5×15.2	CS	1975	0.6× 8.5	ASDN	43	0.27	3.40	B
25	7.3×24.4	CP	1975	0.6×11.0	BSFDN	30	0.08	2.78	B
26[7]	7.3×23.2	CP	1975	0.6× 9.4	BSFDE	168	1.04	1.36	E
27	5.5×18.3	CS	1975	1.2× 8.5	ADCN	168	1.03	1.58	D
28	4.9×15.2	CS	1975	0.9× 6.7	ADN	18	0.24	2.54	B

†[1]Last observed settlement and tilt after first loading. Instrumentation destroyed October 1977.

[2]Structure built in 1974, but full load applied after second year.

[3]According to owner, loads applied were only dead structural load in first year, increasing gradually to maximum nominal load in fourth year. Factor of safety is applied to last load.

[4]Structure straightened after third loading.

[5]Last observed settlement and tilt before third loading; instrumentation destroyed November 1978.

[6]Last observed settlement and tilt before fourth loading; instrumentation destroyed November 1978.

[7]Last observed settlement and tilt on second loading; structure straightened afterwards.

‡CS, precast concrete stave; CP, cast in-place concrete.

§A, annular foundation with small concrete wall at base of tower silo; B, plain annular foundation; H, modification of foundation of previous wooden silo; S, steel reinforcement in footing; F, concrete floor at bottom of silo; D, drainage provided for silage juice; C, cracked foundation (visual observation); N, natural soil, undisturbed; construction may or may not be on a small fill; E, excavation of top part of natural soil; excavation usually less than thickness of annular foundation.

Table 5.2: Reprinted from Morin and Bozozuk 1983
Performance of concrete tower silos on clay in Quebec.
Canadian Agricultural Engineering, Vol. 25, No. 1,
p. 81-88.

TABLE II. PERFORMANCE CRITERIA FOR TOWER SILOS

Rating	Performance	Vertical settlement (mm)	Tilt	
			Degrees	% height
A	Excellent	Below 25	Below 0.2	0.3
B	Good with light problems	25–75	0.2–0.5	0.3–0.8
C	Important problems	75–150	0.5–1.0	0.8–1.7
D	Serious problems	150–300	1.0–1.5	1.7–2.5
E	Very serious problems	Over 300	Over 1.5	Over 2.5

TABLE 5.3

Contact pressures, bearing capacities and safety factor data for 30 tower silos.

#	reported safety factor	round theory contact pressure kPa	calculated bearing capacity kPa	composite footing theory: contact pressure ring kPa	contact pressure floor kPa	safety factor ring	safety factor floor	% change in safety factor ring %	% change in safety factor floor %
1	2.16	97	210	150	69	1.40	3.04	-35.2	+40.7
2	2.62	114	299	182	79	1.64	3.78	-37.4	+44.3
3	3.61	91	329	129	79	2.55	4.16	-29.4	+15.2
4	2.50	148	370	299	79	1.24	4.68	-50.4	+87.2
5	1.68	82	138	110	79	1.25	1.75	-34.4	+4.2
6	2.36	79	186	109	72	1.71	2.58	-27.5	+9.3
7	3.04	91	277	140	57	1.98	4.86	-34.9	+59.9
8	1.80	73	131	101	67	1.30	1.96	-27.8	+8.9
9	2.31	145	335	268	77	1.25	4.35	-45.9	+88.3
10	3.32	98	325	148	70	2.20	4.64	-33.7	+39.6
11	3.52	73	257	101	67	2.54	3.84	-27.8	+9.1
12	1.80	82	148	120	59	1.23	2.51	-31.7	+39.4
13	1.73	102	176	149	84	1.18	1.76	-31.8	+1.7
14	2.42	125	302	209	82	1.44	3.68	-40.5	+52.1
15	2.29	135	309	224	79	1.38	3.91	-39.7	+70.7
16	4.03	114	459	195	80	2.35	5.74	-41.7	+42.4
17	2.59	108	280	166	79	1.69	3.54	-34.7	+36.7
18	1.61	66	106	84	79	1.26	1.34	-21.7	-16.8
19	1.73	105	182	171	56	1.06	3.25	-38.7	+87.9
20	3.28	155	508	314	57	1.62	8.91	-50.6	+171.6
21	1.84	92	169	141	74	1.20	2.28	-34.8	+23.9
22	5.22	92	480	140	71	3.43	6.76	-34.3	+29.5
23	3.52	77	271	105	74	2.58	3.66	-11.0	+4.0
24	3.40	73	248	102	67	2.43	3.71	-28.5	+9.1
25	2.78	145	403	225	105	1.79	3.84	-35.6	+38.1
26	1.36	185	252	355	106	0.71	2.38	-47.8	+75.0
27	1.58	104	164	155	79	1.06	2.08	-32.9	+31.6
28	2.54	97	246	161	63	1.53	3.90	-39.8	+53.5
29	3.00	119	357	163	114	2.19	3.13	-37.0	+4.3
30	1.20	201	241	466	79	0.52	3.05	-126.7	+154.2

pressure. The percentage change in safety factor is also provided. The Hammond and Richmond silos are assigned numbers 29 and 30 respectively.

In all 30 cases considered, composite footing theory predicts a much lower safety factor under the ring footing than does traditional round footing theory (Table 5.3). The average decrease in safety factor for the ring area is 38% going from round to composite theory. In all but one case (no. 18) the lower safety factor under the ring predicted by composite theory is accompanied by a higher safety factor under the floor.

Some of the silos were rated as performing poorly even though the safety factors against bearing capacity, as determined by Morin and Bozozuk (1983), were relatively high. Silos 10 and 11 each had safety factors, calculated with round footing theory, greater than 3.0. Using composite footing theory, the predicted safety factors are 2.20 under the ring for silo number 10 and 2.54 under the ring for silo number 11. This converts to reductions in safety factor of 34% and 28% respectively for silos 10 and 11. Silo number 17 had a reported safety factor of 2.59, but settlement and tilt was unacceptable. With composite footing theory, the safety factor was found to be only 1.69, representing a 35% decrease in safety factor under the heel of the ring. Three more silos with expected safety factors greater than 2.0 also performed poorly. Numbers 1, 9, and 14 had reductions in safety factor under the ring footing of 35%, 46%, and 41% respectively when composite footing theory was applied.

The Richmond Silo (no. 30, Bozozuk 1979) is an example of a catastrophic bearing capacity failure. Reported safety factor from round footing theory was 1.20. Failure can be expected when the applied bearing pressure is close to or exceeds the bearing capacity of the supporting soil. However, composite footing theory suggests this ratio to be 0.52, which is a positive indication that the foundation is underdesigned.

5.2 Subgrade Stresses

The particular example used to illustrate the computer programs pressure bulb solving function is the Hammond, Ontario Silo (see Bozozuk 1979). This site was chosen as an example because the dimensions and loads are precisely known. The dimensions and loading information were loaded into the computer and the program run under various control strategies to illustrate its operating characteristics.

Figure 5.1 shows individual pressure bulbs due to ring loading and floor loading. Reference pressure (P) refers to the intensity of the uniformly distributed load acting on the floor area of the silo. The extent of the interaction between the two sets of isobars can be seen. Only the 20% and 40% bulbs from the ring loading are shown for clarity. Notice the interaction of the two sets of isobars near the heel of the footing. At this location, the additive influences from the ring and floor areas is most pronounced. Since the width of the ring footing is less than the diameter of the silo floor, depth of stress penetration due to ring loading is significantly less.

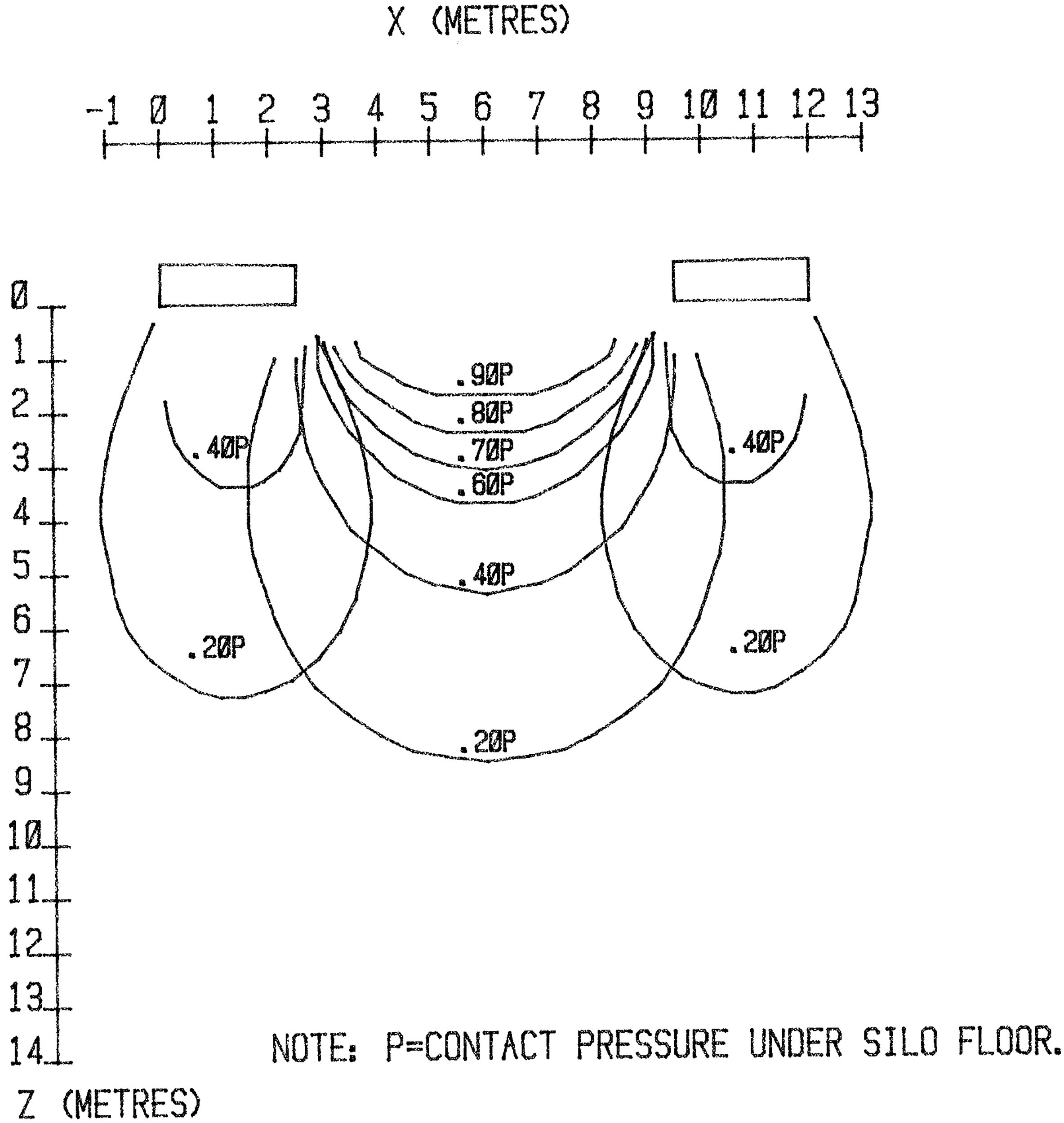


FIG. 5.1: INDIVIDUAL PRESSURE BULBS FOR RING FOOTING AND SILO FLOOR AREA. (HAMMOND ONT. BOZOZUK 1979)

Figure 5.2 compares stress distribution resulting from the round and composite footing models. The round footing model has the entire silo load evenly distributed over the base of the foundation. The composite footing model portions the load between the doughnut footing and the silo floor. Reference pressure (P) is taken as the silo floor load from the composite footing model.

In the region immediately below the footing, stress calculations in accordance with composite footing theory indicate similar stress levels to round footing theory (90% isobar). This is due to the fact that contact stress under the floor is about 105 kPa for both composite and round footing models. As shown in Table 5.3, estimates of contact pressures are higher when composite footing theory is used. Ring loading and floor loading both contribute to stress increase by the principle of superposition. The larger the diameter of the loaded area, the deeper the penetration of significant stressing. This last point is illustrated by Figure 5.2 where all except the 90% isobars reach deeper when round footing theory is employed. This result is due to the relative sizes of the loaded areas. For round footing theory, the characteristic footing dimension is the outer diameter of the ring. This extends the isobars deeper than does composite footing theory because at greater depths, the ring area has limited influence on stress increase and the characteristic footing dimension becomes the inner diameter of the ring.

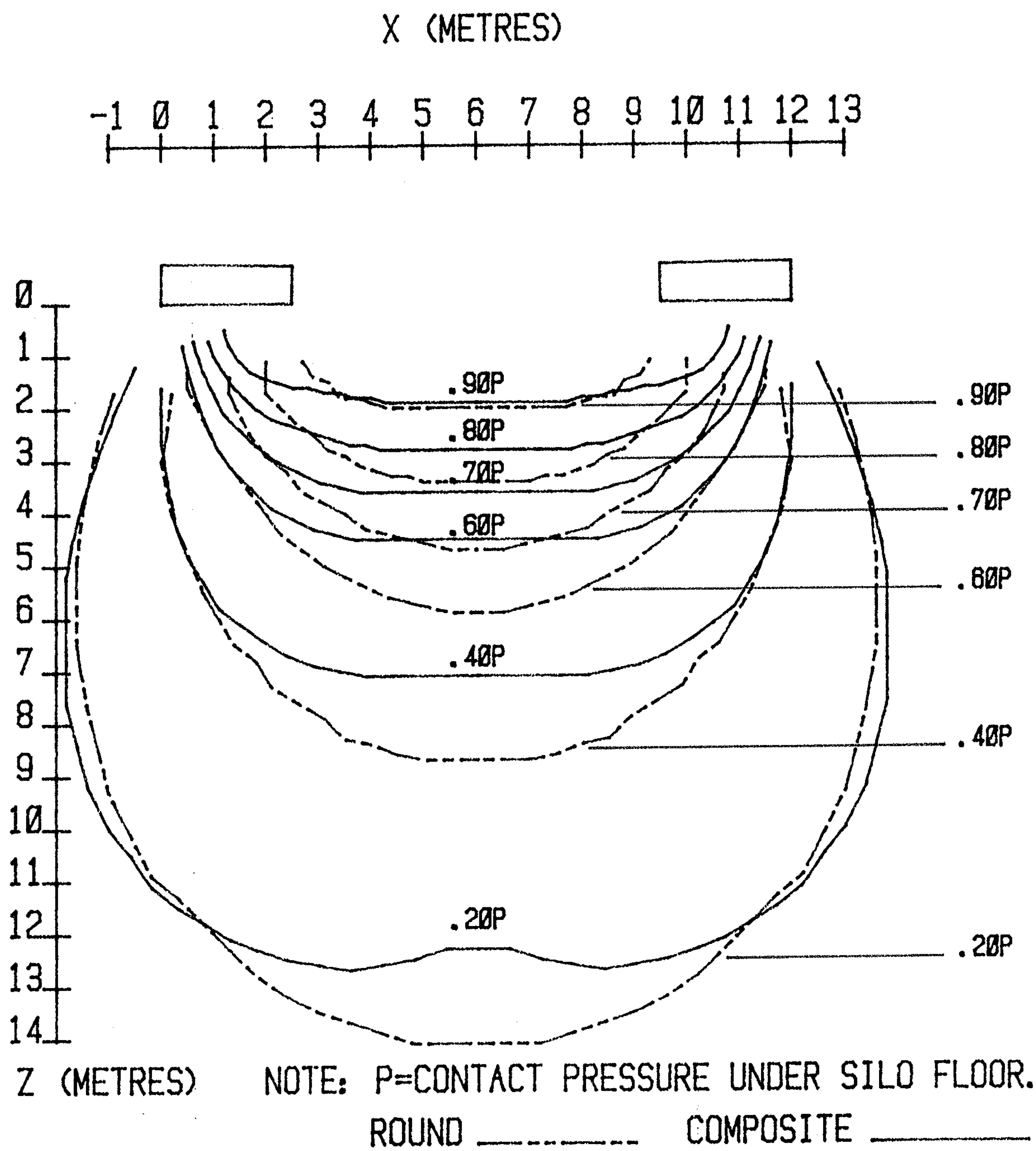


FIG. 5.2: SUPERIMPOSED PRESSURE BULBS FOR ROUND AND COMPOSITE FOOTING MODELS.
(HAMMOND ONT. BOZOZUK 1979)

Figures 5.3, 5.4, and 5.5 are included to show the effect of varying increment sizes in the search grid. Figure 5.2 used 0.100 m for both X and Z increments. This run produced well defined curves but took about 60 h to execute. Figure 5.3 used 0.200 m as X and Z increments. Comparable plots were produced in about one quarter of the time. Figures 5.4 and 5.5 illustrate the effect of having increment size in one direction greater than the other. When the X increment is large, the outer edges of the isobars are poorly defined but the central portions are generally well defined (see fig. 5.4). The opposite is true when the Z increment is large (see fig. 5.5).

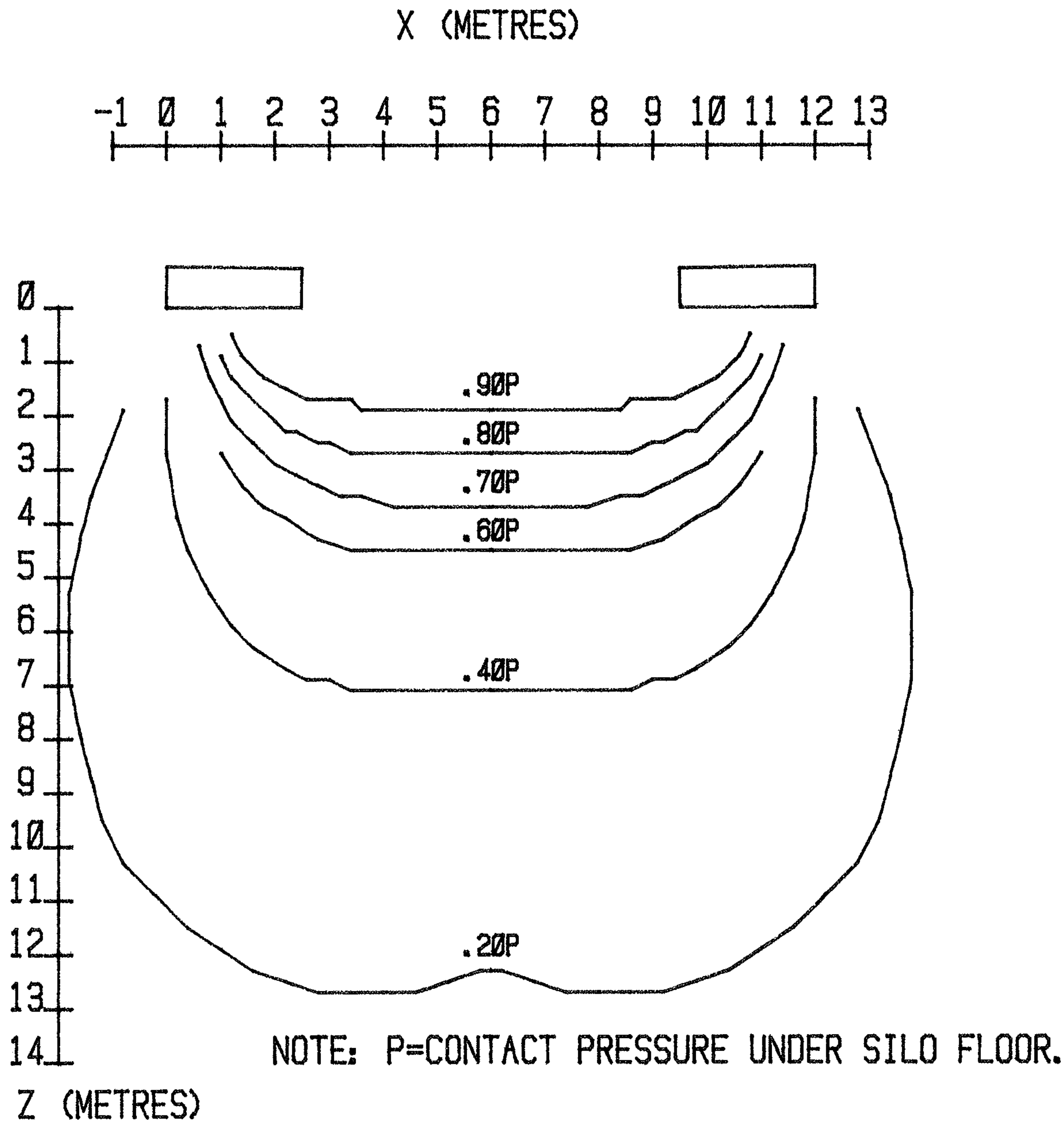


FIG. 5.3: ISOBARS FOR COMPOSITE FOOTING MODEL
INCREMENT SIZES XINC=Ø.2ØØm ZINC=Ø.2ØØm
(HAMMOND ONT. BOZOZUK 1979)

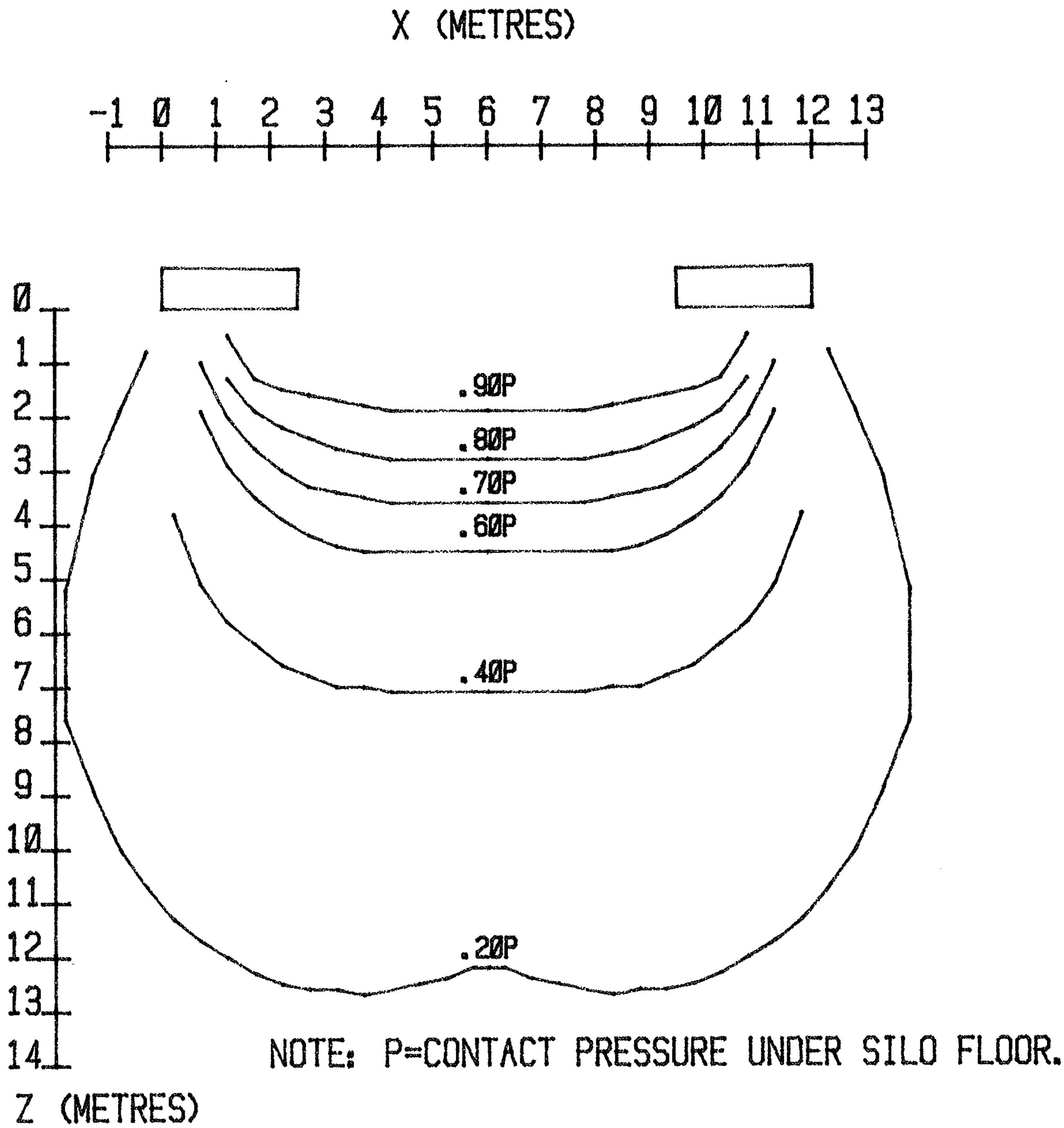
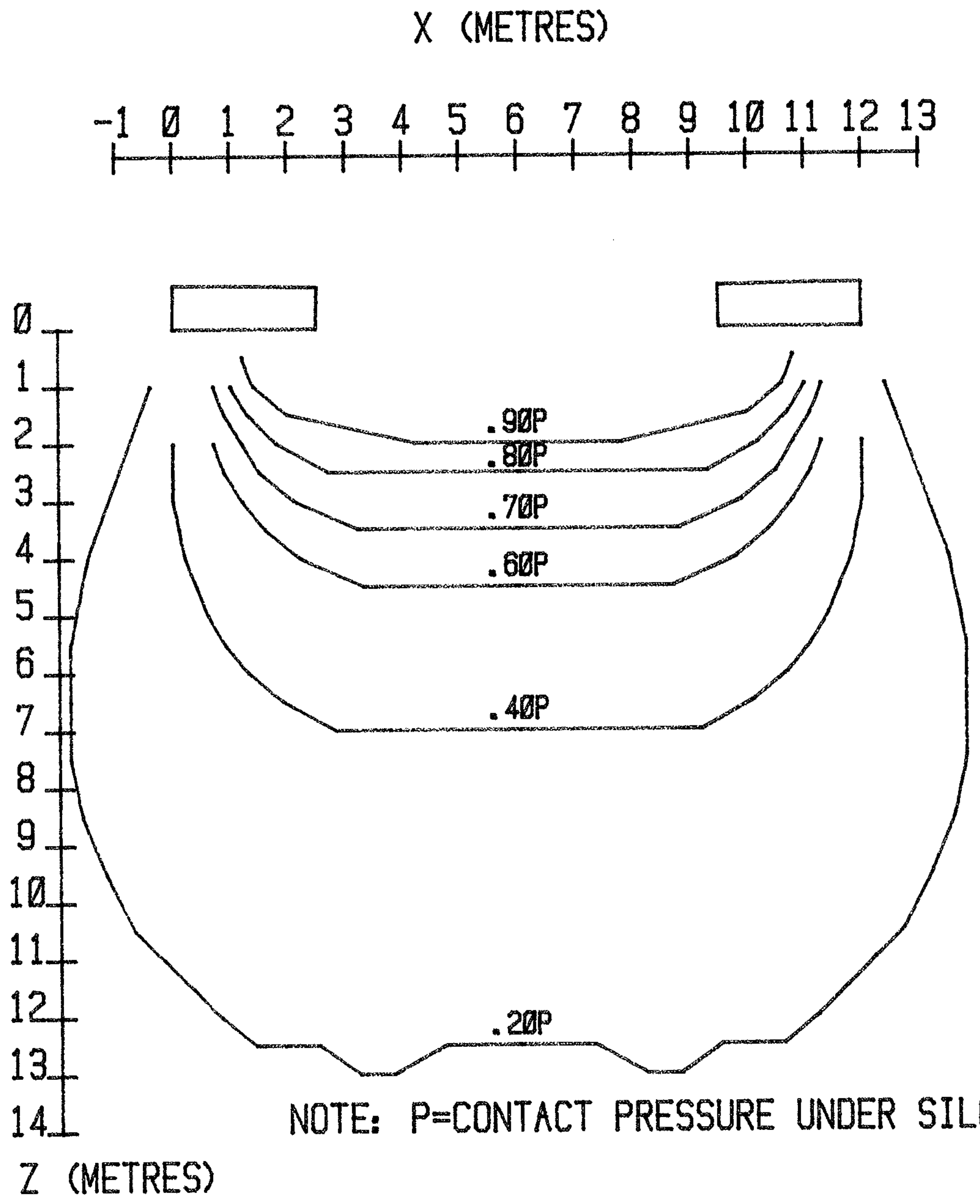


FIG. 5.4: ISOBARS FOR COMPOSITE FOOTING MODEL
INCREMENT SIZES XINC=Ø.5ØØm ZINC=Ø.1ØØm
(HAMMOND ONT. BOZOZUK 1979)



NOTE: P=CONTACT PRESSURE UNDER SILO FLOOR.

FIG. 5.5: ISOBARS FOR COMPOSITE FOOTING MODEL
INCREMENT SIZES XINC=Ø.1ØØm ZINC=Ø.5ØØm
(HAMMOND, ONT. BOZOZUK 1979)

Chapter VI

DISCUSSION

Extended ring tower silo foundations present interesting problems with respect to contact pressure distribution and subgrade stress distribution. The problems arise from the physical properties of this type of foundation, the unique loading conditions present, and the manner in which extended ring tower silo foundations have been observed to perform.

The load transfer theory which is currently applied to tower silo foundations approximates the doughnut shaped footing and silo floor as a single disk shaped slab. This allows soil loading to be calculated as a uniform pressure over the entire base of the foundation. In actuality, the ring footing and silo floor are separate and distinct load transfer bodies. This statement is made on the basis of their individual physical properties, the distinctness of their respective applied loads, and experimental evidence.

Extended ring foundations consist of a reinforced concrete circular mat with a large concentric hole through it. The ring portion of the foundation is fairly rigid due to its thickness, approximately 0.750 m, and its reinforcing steel. The silo floor, previously described as the hole in the ring footing, must certainly be classified as flexible since there is normally no solid interface between the silage mass and the supporting soil. Sometimes a concrete floor is poured separately, but it is usually thin relative to the ring footing. Also, unless this floor is tied-in to the ring with reinforcing steel, it will act independently. Typically, the bearing area of the ring accounts for only 50% to 65% of the total base area of the silo. Thus, it is a gross assumption to consider the foundation base of a tower silo as a solid circular disk.

Further justification for treating the silo foundation as a composite or dual component footing comes from consideration of the nature of the load each bears. As previously discussed, the ring footing component carries a circular line load while the silo floor component carries a uniformly distributed load. The line load is transmitted by the walls and consists of the silo superstructure, mechanical equipment, and silage wall friction. The distributed load on the floor is supplied by the silage mass. Load from these disparate sources determine their associate contact stress. Clearly, contact stress under a ring footing carrying a line load should be expected to differ from the vertical pressure at the silo floor.

Magnitude of the line load carried by the footing and the distributed load carried by the floor are dependent on both the silo properties and the silage properties. Larger silos exert larger contact stress making non-uniformity a cause for greater concern. Cast-in-place concrete silos have thicker, and therefore heavier, walls than the concrete stave silos. This means a cast-in-place silo will place a greater line load on the ring footing. Type, moisture content, and depth of silage all affect the total magnitude and distribution of the silo's contents. For design purposes, this information can be obtained from the Canadian Farm Building Code (1983). The major point about the determination of loads on silo foundations is that the theory required to quantify these loads is readily available. Thus, the justification for making the simplifying solid disk assumption is diminished.

Experimental evidence indicates that over time, and after repeated loadings, vertical pressures at the foundation base tend to approach uniformity (Bozozuk 1979). It is also reported that maximum non-uniformity and maximum peak pressures occur when the silo is first filled to capacity. This non-uniformity is evident both across the width of the ring footing and between the average stress under the ring and under the floor. Engineering practice dictates that foundation design be based on the worst case load conditions likely to be experienced. The worst load conditions exist when the silo is first loaded because pressure non-uniformity and excess porewater pressure is at a maximum. It is known that shear strength of soil is directly related to excess pore pressure. This condition of maximum non-uniformity combined with minimum soil strength also supports the claim that the concrete ring and the silo floor should be considered as separate entities with respect to load transfer, for the purpose of safe foundation design.

The results of this study can be used to explain the "bedding in" phenomenon described by Bozozuk (1979). In his study he describes how the first major loading of an extended ring tower silo caused differential settlement between the inner edge or heel and the outer edge or toe of the ring foundation. This warping of the footing can be explained by localized yielding of the supporting soil which is predicted when the ring and its loading are analysed separately. Separate analysis of the ring under line loading predicts contact stress under the heel of the ring footing which far surpasses the bearing capacity of the soil. Since the area of the footing in compression with the soil is small, only local yielding occurs, causing the differential settlement.

Localized yielding causes the ring foundation to settle in such a way as to more evenly distribute stress. However, due to the large eccentricity of the wall on the ring, the stress distribution is, at least initially, still highly non-uniform. A stress distribution ratio of 2:1 from heel to toe of the footing was chosen, in this study, based on available experimental evidence. Bozozuk (1979 a) found this same ratio to be the maximum observed in his study.

Cohesive foundation soils gain strength as they consolidate. Consolidation involves the expulsion of porewater from the stressed zone. However, this process is time dependent. Therefore, the danger of a bearing capacity failure is at its greatest when the structure first experiences major loading. It has also previously been noted that maximum non-uniformity and maximum peak contact pressures occur during the first major loading. The situation confronting the foundation designer is that the highest contact pressures are exerted by the structure when the supporting soil is in its weakest state. The prudent response is to design the foundation based on a safety factor against bearing capacity failure which reflects the load and soil conditions at their most critical time. That is, when the silo is first loaded non-uniformity is greatest. This can be done by analysing the ring and floor components of the foundation separately as described in chapter 3.

The comparison of classical round footing theory to the proposed composite footing approach to silo foundation design raises some interesting points. Table 5.3 shows that the greatest pressure applied to the supporting soil is located under the heel of the ring footing. Present practice suggests that contact stress under the ring should dictate the foundation design if maximum load is situated there. This conclusion follows implicitly from Turnbull et al. (1979) who describes extended ring foundations as not "wasting unnecessary foundation concrete and reinforcing steel under the silo floor". Often, the cause of silo foundation failure is breakup or cracking of the footing (Bozozuk 1979 b). This is normally due to excess deformation. Again the emphasis is on contact stress under the ring. Bozozuk (1974) also notes that silos usually fail monolithically. However, this does not imply silos are monoliths and hence supply a uniform pressure to the soil. Previous work indicates that the contact pressure is non-uniform. It is not difficult to imagine a monolithic failure being initiated by localized yielding under a portion of the ring footing. It would therefore seem prudent to base bearing capacity analysis on the greatest pressure. Since little is known about pressure distribution across the footing width, if maximum pressure is predicted under the ring, the safest approach is to use the maximum pressure which is found under the heel.

The results of this study hold various implications for the tower silo industry. When composite footing theory is applied to bearing capacity analysis, we find the intensity of loading is significantly greater than that predicted using simple round footing theory. This may explain why some tower silos perform poorly even though they seem to have an adequate safety factor (i.e. greater than 3.0, see tables 5.1 and 5.3 numbers 10 and 11). The use of the composite footing model in design would influence the construction because larger footings would be required to maintain adequate safety factors. Where maximum contact pressure occurs under the ring footing, the footing would have to be sized accordingly. Engineering judgement will be exercised in situations where contact stress under the heel of the footing is extremely high relative to the floor area stress. It may be that a better solution than increasing the outer diameter of the ring would be to decrease the inner diameter to reduce eccentricity and get a more uniform stress distribution. In any case, it is always in the best interest of all concerned for the designer of a silo foundation to have the most accurate and detailed loading information possible. This leads to the safest and most efficient design.

Does significant improvement in accuracy of prediction of subgrade soil pressure result from the use of composite footing theory? The answer is difficult to find directly because experiments to measure soil pressure over such a wide area have not been conducted. A promising method of comparing the two theories is by settlement calculation. The more accurate method for determining stress increase would lead to the more accurate settlement predictions in the majority of cases. Unfortunately, insufficient information was available for this worthwhile investigation to be undertaken by this study. Indications from the contact stress portion of the study are that the composite footing model is an improvement over the round footing model. Logical progression suggests that if contact stress prediction is more accurate, then subgrade stress increase predictions will also be more accurate.

Prediction of stress increase under a footing is critically dependent upon the manner in which the footing is modeled. This is clearly evident in Figure 5.2. When an extended ring silo foundation is modeled as a simple round mat instead of a ring and floor area composite, predicted stress increase is found to be lower in the zone beneath the footing. This is an important result because, for homogeneous soils, the largest settlements occur in the upper regions of the founding soil which is also the most highly stressed.

Predicting settlement of tower silos is complicated by the problem of accurately determining stress increase in the supporting soil and the intermittent cyclic nature of the loading. With the use of a computer program which can, relatively quickly, solve for stress increase, these problems may be circumvented. By running a series of trials for a given silo using various depths of silage, a record of stress increase with respect to loading could be made. This information would allow settlement calculations to be made for the time interval a given load is present. This method could be useful as an analytical tool. However, it is normally impossible to accurately predict the magnitude and duration of loading a silo will experience. Nevertheless, an estimate can be made, in consultation with the silo owner, which could yield a useful prediction of potential settlement. Designers would then have the welcome addition of settlement information to supplement bearing capacity analysis. Presently, silo foundations are designed on the basis of bearing capacity alone.

The computer program defines stress increase isobars whose accuracy depends on the increment or step sizes used in working through the search grid. To get well defined curves for Figure 5.2, both X and Z increment sizes were 0.100 m. This program took approximately 60 h to execute. Acceptable results can be had with much shorter execution times by increasing the increment size. Figure 5.3 results from using the suggested increment of 0.200 m for both X and Z directions. Estimated execution time is 15.3 h in this case. Although it would normally be inappropriate to allow a single problem to monopolize over 15 h of computer time, running the program overnight is probably acceptable.

Choice of increment size depends on the length of execution time the user is willing to accept and also the specifics of the information the user requires. A large X increment, say 0.500 m, and a small Z increment of 0.100 m will clearly define the central portions of the isobars but will be less accurate at the outer edges. Similarly, an X increment of 0.100 m and Z increment of 0.500 m would more clearly define the edges of the isobar but give somewhat more erratic results towards the centre (Figures 5.4 and 5.5). Execution time for the program with these increment sizes is about 12 h.

Often the user will only be interested in detailed stress increase information under a particular part of the silo. For this instance, the X increment should be such that the desired location is reached in one X step to reduce execution time. If the Z increment was 0.200 m, execution time would be about 22 min to complete a single point vertical scan under the silo. With judicious selection of input variables the program lends itself to practical usage. It should also be noted that if a program were available to compile the BASIC code into machine code, execution times could be reduced to somewhere between one quarter to one twenty-fifth their present length.

A further useful feature of the program is the calculation of contact pressure for either the composite or round footing models. This is a user interactive function of the program which allows various silo dimensions and depths of silage to be entered and returns an estimate of the corresponding contact pressures.

Chapter VII

CONCLUSION

The following conclusions have been derived from the preceding study:

1. Based upon the study of 30 silos, extended ring tower silo foundations are more appropriately modelled as a two piece, ring and floor area composite footing, rather than a single round disk.

2. Loads imposed by tower silos on their foundations can be reasonably well determined using existing silage pressure theory. A line load is transmitted by the silo walls to the ring foundation and a uniformly distributed load is present on the floor area.

3. The two component surface loading can be considered as a linearly increasing distributed load in a 2:1 ratio, heel to toe, across the ring footing and a uniformly distributed load over the silo floor area.

4. Composite footing theory consistently predicts higher contact pressures than does present round footing theory. This leads to a reduction in calculated safety factor against bearing capacity failure.

5. An appropriate safety factor can be calculated on the basis of the highest contact pressure as derived from composite footing theory.

6. The computer program which the author developed to model an extended ring silo foundation and its loading, produces stress distribution diagrams, and calculates foundation loading data which can be utilized for detailed foundation design.

7. The computer program allows the user to control the subgrade stress prediction precision and the length of execution time.

Chapter VIII

RECOMMENDATIONS FOR FURTHER STUDY

1. Accurate experimental data for pressure under the ring footing and silo floor area during and immediately after the silo is first loaded is required to verify pressure distribution assumptions. Information is particularly required on the distribution across the width of the footing.

2. Forces on the ring should be analyzed, possibly by finite element, to verify the assumption that a ring footing can be modelled as a strip.

3. Settlement calculations should be made using the computer program to solve for stress increase. These results could then be compared with the corresponding observed settlements to evaluate the accuracy of the model.

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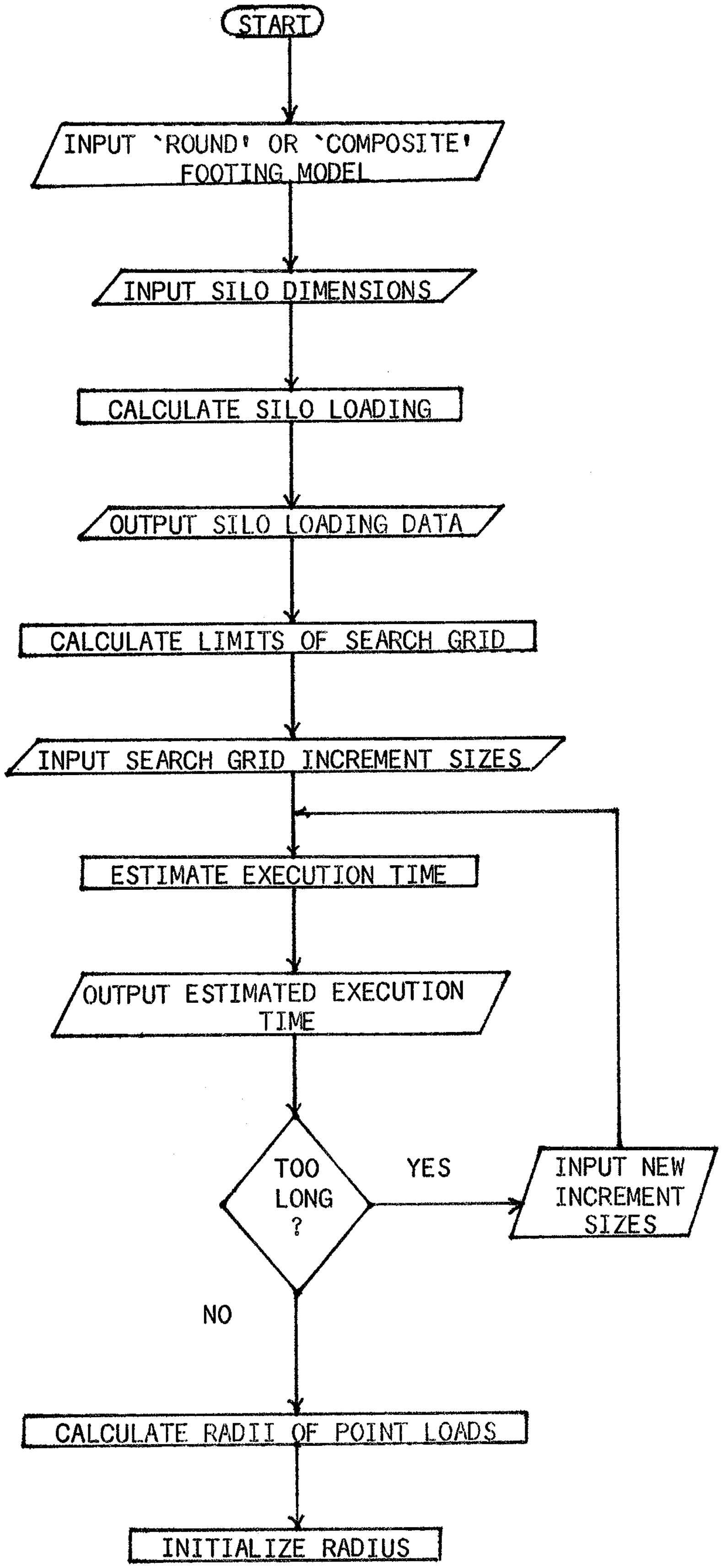
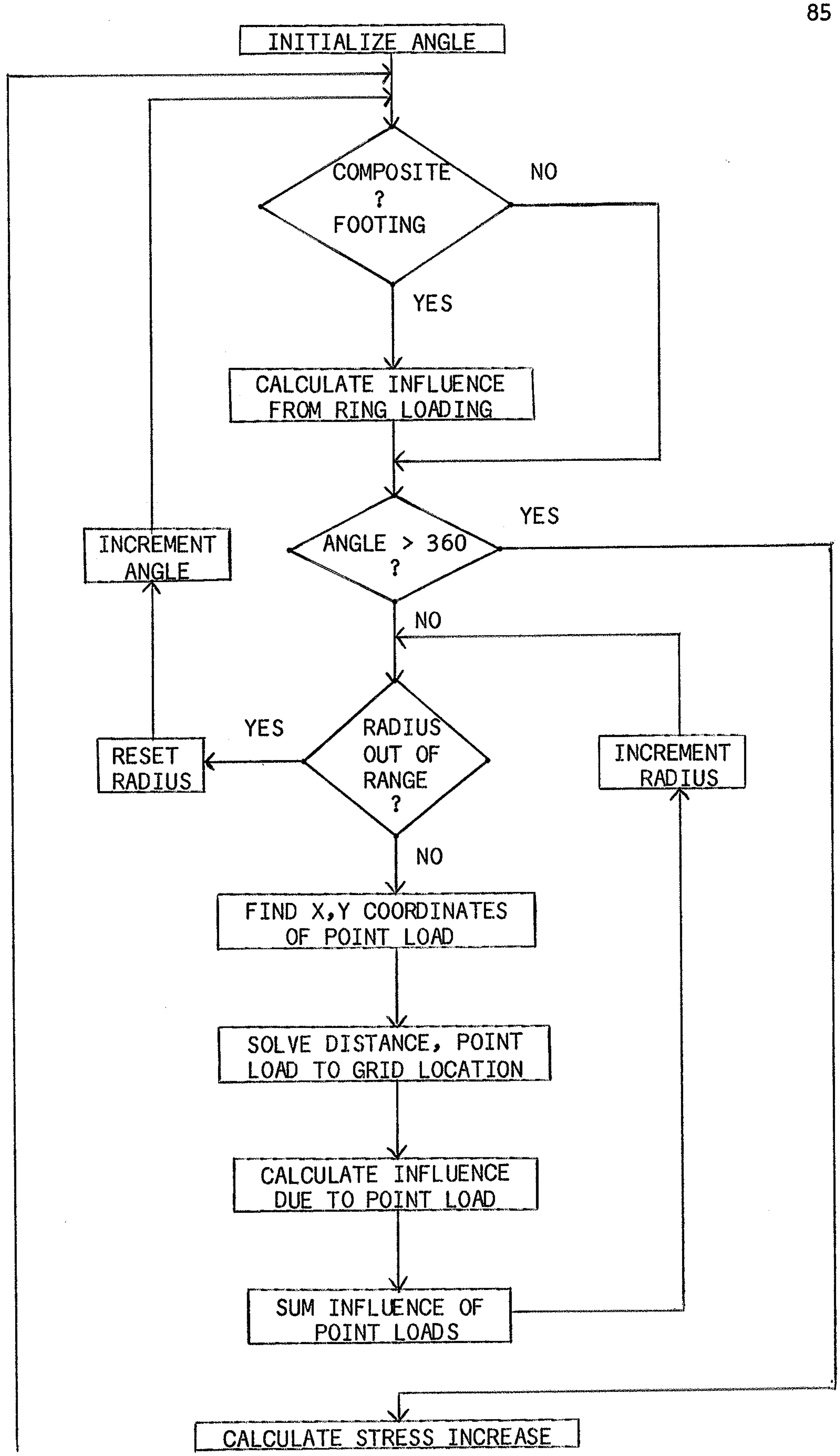
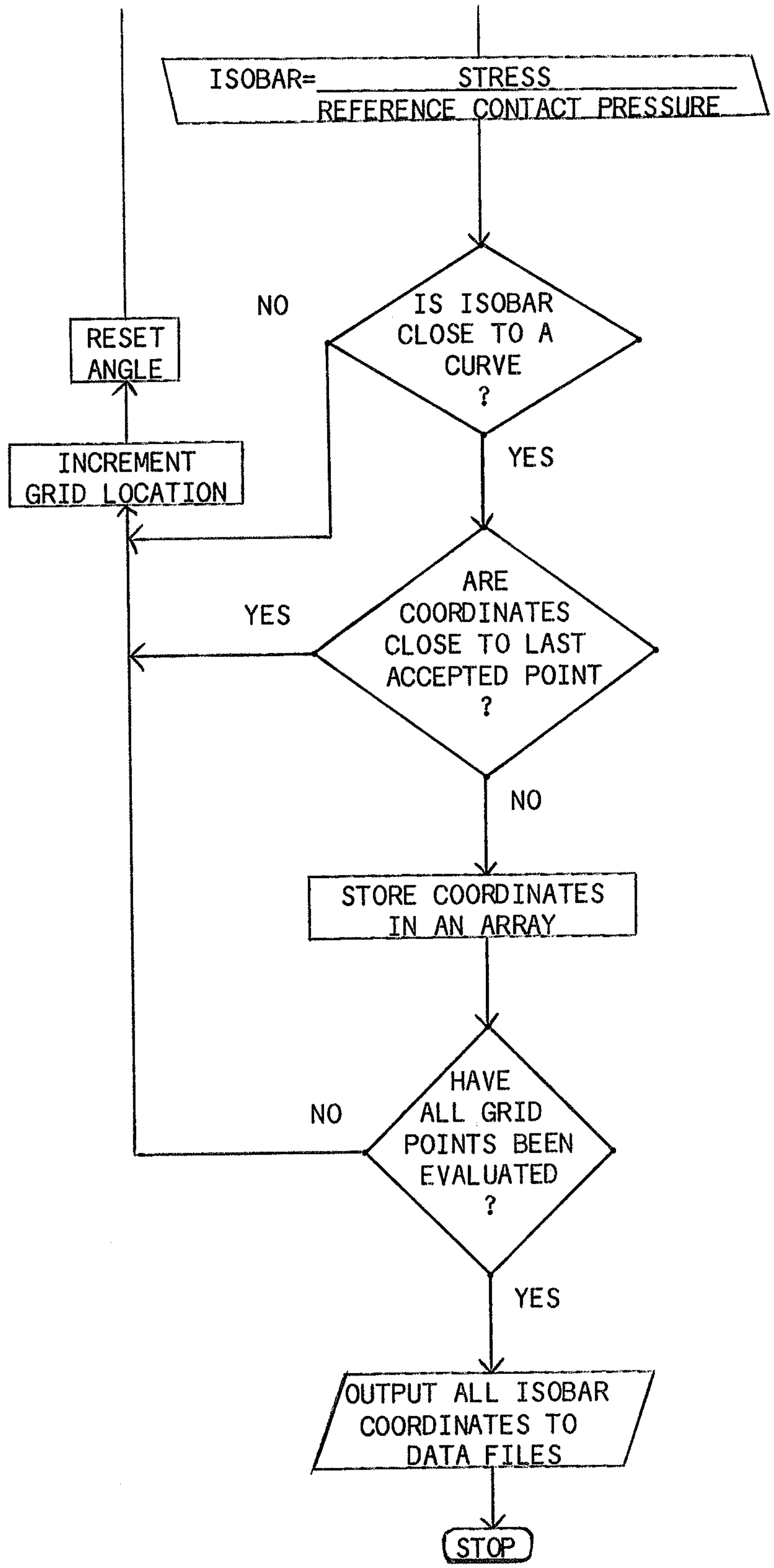
Appendix A

FLOWCHART OF MAIN PROGRAM


START
INPUT `ROUND' OR `COMPOSITE' FOOTING MODEL
INPUT SILO DIMENSIONS
CALCULATE SILO LOADING
OUTPUT SILO LOADING DATA
CALCULATE LIMITS OF SEARCH GRID
INPUT SEARCH GRID INCREMENT SIZES
ESTIMATE EXECUTION TIME
OUTPUT ESTIMATED EXECUTION TIME
TOO LONG ?
YES
INPUT NEW INCREMENT SIZES
NO
CALCULATE RADII OF POINT LOADS
INITIALIZE RADIUS


INITIALIZE ANGLE
COMPOSITE ? FOOTING
NO
YES
CALCULATE INFLUENCE FROM RING LOADING
ANGLE > 360 ?
YES
INCREMENT ANGLE
NO
RADIUS OUT OF RANGE ?
YES
RESET RADIUS
INCREMENT RADIUS
NO
FIND X,Y COORDINATES OF POINT LOAD
SOLVE DISTANCE, POINT LOAD TO GRID LOCATION
CALCULATE INFLUENCE DUE TO POINT LOAD
SUM INFLUENCE OF POINT LOADS
CALCULATE STRESS INCREASE


ISOBAR= STRESS / REFERENCE CONTACT PRESSURE
IS ISOBAR CLOSE TO A CURVE ?
NO
YES
RESET ANGLE
INCREMENT GRID LOCATION
ARE COORDINATES CLOSE TO LAST ACCEPTED POINT ?
YES
NO
STORE COORDINATES IN AN ARRAY
HAVE ALL GRID POINTS BEEN EVALUATED ?
NO
YES
OUTPUT ALL ISOBAR COORDINATES TO DATA FILES
STOP

Appendix B

MAIN PROGRAM LISTING

```
10 '  RESULT.BAS
20 '
30 '        ***********************************************************
40 '        * BOUSSINESQ NUMERICAL INTEGRATION OF ROUND FOOTING ISOBARS *
50 '        * OPTIONAL COMBINED WITH INFLUENCE FROM RING FOOTING.       *
60 '        ***********************************************************
70 '
80 '
90 REM   UNIVERSITY OF MANITOBA, AGRICULTURAL ENGINEERING DEPARTMENT
100 REM   PROGRAMMED BY M. D. VANDERPONT
110 REM   LAST UPDATED JULY 14, 1984
120 '
130 PRINT "PROGRAM TO SOLVE FOR COORDINATES OF PRESSURE ISOBARS UNDER A TOWER SI
LO FOUNDED ON AN EXTENDED RING FOOTING"
140 REM SURFACE LOADING CAN BE EITHER UNIFORMLY DISTRIBUTED OVER ENTIRE
        FOUNDATION BASE OR A COMBINATION OF AN UNIFORMLY DISTRIBUTED LOAD ON
       FLOOR PLUS A RING LOAD ON THE EXTENDED RING FOOTING.
150 PRINT " "
160 PRINT "( 1 )  `ROUND' FOOTING MODEL ASSUMES ENTIRE SILO LOAD IS UNIFORMLY DI
STRIBUTED         OVER FOUNDATION BASE"
170 PRINT "( 2 )  `COMPOSITE' FOOTING MODEL ASSUMES SILO LOAD IS PROPORTIONED BE
TWEEN             CONCRETE RING AND SILO FLOOR"
180 PRINT " "
190 INPUT "INPUT (1) FOR `ROUND' FOOTING MODEL , (2) FOR `COMPOSITE' FOOTING MOD
EL";MODEL
200 IF MODEL=1 THEN PRINT "ROUND FOOTING MODEL" ELSE PRINT "COMPOSITE FOOTING MO
DEL"
210 PRINT TIME$
220 PRINT "ALL DIMENSIONS IN SI UNITS : LENGTH IN METRES"
230 INPUT "OUTER DIAMETER OF RING FOOTING";OD
240 INPUT "INNER DIAMETER OF RING FOOTING";ID
250 INPUT "RING FOOTING THICKNESS";RTH
260 INPUT "SILO DIAMETER";SD
270 INPUT "SILO HEIGHT";SH
280 INPUT "WALL THICKNESS";WTH
290 INPUT "DEPTH OF SILAGE";DEPTH
300 PRINT " " : PRINT " "
310 PRINT USING "FOOTING EXTERIOR DIAMETER IS          ##.### METRES";OD
320 PRINT USING "FOOTING INTERIOR DIAMETER IS          ##.### METRES";ID
330 PRINT USING "SILO DIAMETER IS                      ##.### METRES";SD
340 WIDE=(OD-ID)/2 : H=WIDE
350 PRINT USING "WIDTH OF RING FOOTING IS      ##.### METRES";H
360 '
370 PI=3.141593
380 GOSUB 1580  '  SUBROUTINE CALCULATES FOUNDATION LOADING
390 IF MODEL=1 THEN PRINT USING "REFERENCE CONTACT PRESSURE IS ### kPa";REFSTRES
S : PRINT USING "TOTAL BEARING AREA OF BASE IS ###.# m^2";TAREA : GOTO 470
400 IF REFSTRESS=RINGMAX GOTO 420
410 PRINT USING "REFERENCE CONTACT PRESSURE IS ### kPa";REFSTRESS : GOTO 430
420 PRINT USING "REFERENCE CONTACT PRESSURE IS ### kPa UNDER RING";REFSTRESS
430 PRINT USING "UNIFORMLY DISTRIBUTED FLOOR LOADING IS ### kPa";FLOOR
440 PRINT USING "WALL LINE LOAD IS ### kN/m";WALL
450 PRINT USING "TOTAL BEARING AREA OF SILO FLOOR IS  ###.# m^2";FAREA
460 PRINT USING "TOTAL BEARING AREA OF RING IS ###.# m^2";RAREA
470 REM  DIVIDE INTO APPROPRIATE SECTIONS
480 OPTION BASE 1
490 DIM I2(50,3),I4(50,3),I6(50,3),I7(30,3),I8(30,3),I9(30,3),RC(10)
500 GOSUB 2110
510 '
520 '
530 REM              ***** CONTROL PARAMETERS FOR CALCULATION LOOP *****
540 PRINT " "
550 PRINT "INPUT STEP SIZE FOR SEARCH GRID ; SUGGEST INCX=.2 , INCZ=.2"
560 INPUT "GRID STEP SIZE X-DIRECTION";INCX
570 INPUT "GRID STEP SIZE Z-DIRECTION";INCZ
580 REM  ROUND OFF XSTART TO NEAREST TENTH
```

```
590 XSTART=-.2*OD/2 : XSTOP=OD/2 : ZSTART=.5 : ZSTOP=1.1*OD 'SIZE OF SEARCH GRID

600 TEMP%=XSTART*10 : XSTART=TEMP%/10
610 XSCALE=XSTOP-XSTART
620 ET=((XSTOP-XSTART)/INCX*(ZSTOP-ZSTART)/INCZ*24.5)/3600
630 PRINT USING "ESTIMATED EXECUTION TIME ##.# HOURS";ET
640 PRINT "IS EXECUTION TIME ACCEPTABLE ? : Y/N"
650 ANS$=INKEY$ : IF ANS$="" THEN 650
660 IF ANS$="Y"OR ANS$="y" THEN GOTO 670 ELSE PRINT "RESET STEP SIZE" : GOTO 560

670 '
680 PRINT "PROGRAM RUNNING"
690 DEFINT A,L
700 REM  360 DEGREES IS DIVIDED INTO 20 , 18 DEGREE PIES.
710 REM  EACH PIE IS SUBDIVIDED INTO 10 EQUIVALENT AREAS FOR A TOTAL OF 200
         ELEMENTS
720 '
730 REM                    *****    CALCULATION LOOP    *****
740 '
750 FOR PX=XSTART TO XSTOP STEP INCX
760   FOR PZ=ZSTART TO ZSTOP STEP INCZ
770 IF MODEL=1 GOTO 810
780     BETA=ATN((PX-H)/PZ)
790     ALPHAI=ATN(PX/PZ)-BETA
800     RINGINFL=((PX/H*ALPHAI-SIN(2*BETA)/2)+(ALPHAI+SIN(ALPHAI)*COS(ALPHAI+2*BE
TA)))/PI
810      FOR ANGLE=9 TO 351 STEP 18
820        RANGLE=ANGLE*PI/180
830        FOR L=1 TO 10
840           CX=RC(L)*COS(RANGLE) : CY=RC(L)*SIN(RANGLE)
850           R=SQR((XSCALE+CX)*(XSCALE+CX)+CY*CY+PZ*PZ)
860           INFL=(1.5*PZ*PZ*PZ)/(PI*R*R*R*R*R) : SUMINFL=SUMINFL+INFL
870        NEXT L
880      NEXT ANGLE
890      STRESS=SUMINFL*SPIKE+RINGINFL*W : ' COMMENT SPIKE AND W CONVERTS
                                             INFLUENCE TO STRESS
900      ISOBAR=STRESS/REFSTRESS
910      IF (ISOBAR>.196)AND(ISOBAR<.204) THEN Q=1 : IF Q>0 GOTO 990
920      IF (ISOBAR>.394)AND(ISOBAR<.406) THEN Q=2 : IF Q>0 GOTO 990
930      IF (ISOBAR>.593)AND(ISOBAR<.607) THEN Q=3 : IF Q>0 GOTO 990
940      IF (ISOBAR>.692)AND(ISOBAR<.708) THEN Q=4 : IF Q>0 GOTO 990
950      IF (ISOBAR>.791)AND(ISOBAR<.809) THEN Q=5 : IF Q>0 GOTO 990
960      IF (ISOBAR>.89)AND(ISOBAR<.91) THEN Q=6 : IF Q>0 GOTO 990
970 PRINT USING "    ##.###"; PX,PZ,ISOBAR ' STATEMENT OPTIONAL
980      GOTO 1000
990     ON Q GOSUB 1070,1120,1170,1220,1270,1320
1000    SUMINFL=0 : NEXT  PZ
1010    XSCALE=XSCALE-INCX : NEXT PX
1020 GOTO 1400
1030 '
1040 REM  ***** SUBROUTINES TO LOAD X,Z, COORDINATES & I INFLUENCE COEFF. *****
1050 '
1060 '                      20% ISOBAR
1070 IF ABS(PX-HOLD2X)<.5 AND ABS(PZ-HOLD2Z)<.75 GOTO 1110
1080 IF ABS(PX-HOLD2XX)<.5 AND ABS(PZ-HOLD2ZZ)<.75 GOTO 1110
1090 HOLD2XX=HOLD2X : HOLD2ZZ=HOLD2Z : HOLD2X=PX : HOLD2Z=PZ
1100 C2=C2+1 : I2(C2,1)=PX : I2(C2,2)=PZ : I2(C2,3)=ISOBAR : PRINT PX,PZ,ISOBAR,
"**ACCEPTED**"
1110 RETURN
1120 '                      40% ISOBAR
1130 IF ABS(PX-HOLD4X)<.4 AND ABS(PZ-HOLD4Z)<.4 OR PZ<HOLD4Z GOTO 1160
1140 HOLD4X=PX : HOLD4Z=PZ
1150 C4=C4+1 : I4(C4,1)=PX : I4(C4,2)=PZ : I4(C4,3)=ISOBAR : PRINT PX,PZ,ISOBAR,
"**ACCEPTED**"
1160 RETURN
```

```
1170 '                        60% ISOBAR
1180 IF ABS(PX-HOLD6X)<.3 AND ABS(PZ-HOLD6Z)<.3 OR PZ<HOLD6Z GOTO 1210
1190 HOLD6X=PX : HOLD6Z=PZ
1200 C6=C6+1 : I6(C6,1)=PX : I6(C6,2)=PZ : I6(C6,3)=ISOBAR : PRINT PX,PZ,ISOBAR,
"**ACCEPTED**"
1210 RETURN
1220 '                        70% ISOBAR
1230 IF ABS(PX-HOLD7X)<.3 AND ABS(PZ-HOLD7Z)<.3 OR PZ<HOLD7Z GOTO 1260
1240 HOLD7X=PX : HOLD7Z=PZ
1250 C7=C7+1 : I7(C7,1)=PX : I7(C7,2)=PZ : I7(C7,3)=ISOBAR : PRINT PX,PZ,ISOBAR,
"**ACCEPTED**"
1260 RETURN
1270 '                        80% ISOBAR
1280 IF ABS(PX-HOLD8X)<.2 AND ABS(PZ-HOLD8Z)<.3 OR PZ<HOLD8Z GOTO 1310
1290 HOLD8X=PX : HOLD8Z=PZ
1300 C8=C8+1 : I8(C8,1)=PX : I8(C8,2)=PZ : I8(C8,3)=ISOBAR : PRINT PX,PZ,ISOBAR,
"**ACCEPTED**"
1310 RETURN
1320 '                        90% ISOBAR
1330 IF ABS(PX-HOLD9X)<.2 AND ABS(PZ-HOLD9Z)<.3 OR PZ<HOLD9Z GOTO 1360
1340 HOLD9X=PX : HOLD9Z=PZ
1350 C9=C9+1 : I9(C9,1)=PX : I9(C9,2)=PZ : I9(C9,3)=ISOBAR : PRINT PX,PZ,ISOBAR,
"**ACCEPTED**"
1360 RETURN
1370 '
1380 REM         ***** LOAD COORDINATES INTO DATA FILES *****
1390 '
1400 OPEN "COORD2.DAT" FOR OUTPUT AS #1
1410 FOR L=1 TO C2 : PRINT #1,I2(L,1);I2(L,2);I2(L,3) : NEXT L
1420 CLOSE
1430 OPEN "COORD4.DAT" FOR OUTPUT AS #1
1440 FOR L=1 TO C4 : PRINT #1,I4(L,1);I4(L,2);I4(L,3) : NEXT L
1450 CLOSE
1460 OPEN "COORD6.DAT" FOR OUTPUT AS #1
1470 FOR L=1 TO C6 : PRINT #1,I6(L,1);I6(L,2);I6(L,3) : NEXT L
1480 CLOSE
1490 OPEN "COORD7.DAT" FOR OUTPUT AS #1
1500 FOR L=1 TO C6 : PRINT #1,I7(L,1);I7(L,2);I7(L,3) : NEXT L
1510 CLOSE
1520 OPEN "COORD8.DAT" FOR OUTPUT AS #1
1530 FOR L=1 TO C8 : PRINT #1,I8(L,1);I8(L,2);I8(L,3) : NEXT L
1540 CLOSE
1550 OPEN "COORD9.DAT" FOR OUTPUT AS #1
1560 FOR L=1 TO C9 : PRINT #1,I9(L,1);I9(L,2);I9(L,3) : NEXT L
1570 CLOSE : GOTO 2220
1580 '
1590 REM        ***** SUBROUTINE TO CALCULATE FOUNDATION LOADING *****
1600 '
1610 REM   DEFININTION OF VARIABLES
1620 ' WALLVOL=VOLUME OF WALLS OF SILO {m^3}
1630 ' W1=LINE LOAD ON RING FOOTING DUE TO SILO SUPERSTRUCTURE {kN/m}
1640 ' W2=LINE LOAD ON RING FOOTING DUE TO SILAGE FRICTION ON WALLS {kN/m}
1650 ' W3=LINE SOAD ON RING FOOTING DUE TO DIRECT SILAGE LOADING (kN/m)
1660 ' SDEN=SILAGE DENSITY {kN/m^3}
1670 ' TSL=TOTAL SILAGE LOAD {kN}
1680 ' SFL=AMOUNT OF SILAGE LOAD CARRIED BY SILO FLOOR {kN}
1690 ' SWL=AMOUNT OF SILAGE LOAD CARRIED BY SILO WALL {kN}
1700 ' SOIL= WEIGHT OF SOIL FROM SILO FLOOR TO FOUNDATION DEPTH {kN}
1710 ' OVERBURDEN=PRESSURE AT FOUNDATION DEPTH DUE TO SOIL {kPa}
1720 ' STRUCTWT=TOTAL WEIGHT OF THE ENTIRE SILO {kN}
1730 ' SPIKE=MAGNITUDE OF POINT LOADS WHICH ESTIMATES UDL ON ROUND AREA {kN}
1740 ' RINGMAX= 2*W =MAX. INTENSITY OF CONTACT PRESSURE UNDER RING FOOTING {kPa}

1750 IF MODEL=1 THEN RADIUS=OD/2 ELSE RADIUS=ID/2
1760 WALLVOL=SH*PI*((SD/2+WTH)^2-(SD/2)^2)
```

```
1770 W1=23.56*WALLVOL/(PI*SD)*1.023
1780 REM 1.023 ACCOUNTS FOR ROOF AND EQUIPMENT LOAD
1790 REM TSL=TOTAL SILAGE LOAD , SDEN=SILAGE DENSITY   REFERENCE CFBC
1800 IF DEPTH < 10.67 THEN SDEN=6.44 : GOTO 1860
1810 IF DEPTH < 13.72 THEN SDEN=7.38 : GOTO 1860
1820 IF DEPTH < 16.76 THEN SDEN=8.01 : GOTO 1860
1830 IF DEPTH < 19.81 THEN SDEN=8.8 : GOTO 1860
1840 IF DEPTH < 22.86 THEN SDEN=9.43 : GOTO 1860
1850 IF DEPTH > 22.86 THEN SDEN=10.05
1860 TSL=PI*SD^2/4*DEPTH*SDEN
1870 REM SWL=SILAGE WALL LOAD , SFL=SILAGE FLOOR LOAD
1880 SWL=TSL*DEPTH/(4.7*SD)*(1-DEPTH/(14.1*SD))  ' REFERENCE CFBC
1885 W2=SWL/(PI*SD)
1890 W3=(SD^2-ID^2)*DEPTH*SDEN/(4*SD)
1900 WALL=W1+W2+W3
1910 SFL=TSL-SWL-W3*PI*SD
1920 OVERBURDEN=16*(RTH+.5) ' ASSUMES UNIT WEIGHT OF SOIL IS 16 kN/m^3
1930 RAREA=PI/4*(OD^2-ID^2)
1940 FAREA=PI*ID^2/4
1950 TAREA=PI*OD^2/4
1960 CAREA=PI*RADIUS^2
1970 RP=23.56*RTH
1980 IF MODEL=1 THEN GOTO 2040
1990 W=(RP*RAREA+WALL*PI*SD)/(1.5*RAREA) ' MINIMUM PRESSURE UNDER RING
2000 RINGMAX=2*W
2010 FLOOR=SFL/((PI*ID^2)/4)+OVERBURDEN
2020 IF RINGMAX > FLOOR THEN REFSTRESS=RINGMAX ELSE REFSTRESS=FLOOR
2030 GOTO 2080
2040 RINGVOL=RTH*PI/4*(OD^2-ID^2)
2050 STRUCTWT=23.56*(WALLVOL*1.023+RINGVOL)
2060 SOIL=16*PI*ID^2/4*(RTH+.5)
2070 FLOOR=(STRUCTWT+SOIL+TSL)/(PI*OD^2/4) : REFSTRESS=FLOOR
2080 SPIKE=FLOOR*CAREA/200 ' MAGNITUDE OF POINT LOAD ON EACH ELEMENT
2090 RETURN
2100 '
2110 REM  SUBROUTINE CALCULATES CENTROID RADIUS FOR EACH ELEMENT OF ROUND AREA
2120 '            ************************
2130 AREA=CAREA/200
2140 REM 'AREA' IS SIZE OF INDIVIDUAL ELEMENT WHICH CIRCULAR AREA 'CAREA' IS
          SUBDIVIDED INTO
2150 L=1
2160 FOR S=20 TO 200 STEP 20
2170    ROUT=SQR(S*AREA/PI) : RC=SQR((RIN*RIN+ROUT*ROUT)/2) : RC(L)=RC
2180    RIN=ROUT : L=L+1
2190 NEXT S
2200 RETURN
2210 '
2220 BEEP :PRINT "ISOBAR COORDINATES IN DATA FILES;USE PGM GETCOORD.BAS": PRINT
TIME$
2230 PRINT "PRESSURE ISOBARS & INFLUENCE COEFFICIENTS"
2240 PRINT "REFERENCE LOADING IS ",REFSTRESS,"kPa"
2250 IF MODEL=1 THEN PRINT "LOAD UNIFORMLY DISTRIBUTED OVER ENTIRE FOUNDATION BA
SE"
2260 IF MODEL=2 THEN PRINT "COMPOSITE FOOTING SCHEME"
2270 PRINT "************ END *************" : STOP :END
```

Appendix C

PROGRAM TO ACCESS DATA FILES

```
10 REM  GETCOORD.BAS OPENS DATA FILES FOR INSPECTION
20 OPEN "I",#1,"COORD2.DAT"
30 FOR K=1 TO 30
40 INPUT#1, A, B, C
50 PRINT USING "    ##.###";A,B,C
60 NEXT K
70 CLOSE
```