11) Allowable Bearing Pressure in Sand Based on Settlement Consideration

- Meyerhof ➡

For B ≤ 1.22m,

$$q_{net(all)}(kN/m^2) = 11.98(N_1)_{60}$$

For B > 1.22m

$$q_{net(all)}(kN/m^2) = 7.99(N_1)_{60}(\frac{3.28B+1}{3.28B})^2$$ (B in meters)

where $(N_1)_{60}$ is the corrected penetration resistance.

➡ Too conservative.
➡ Later, Meyerhof suggested that $q_{net(all)}$ should be increased by 50%.

- Bowles ➡

For B ≤ 1.22m

$$q_{net(all)}(kN/m^2)=19.16(N_1)_{60}F_d(S/25.4)$$

For B > 1.22m

$$q_{net(all)}(kN/m^2) = 11.98(N_1)_{60}(\frac{3.28B+1}{3.28B})^2 F_d(\frac{S}{25.4})$$

where, $F_d = depth\ \ factor = 1+0.33(D_f/B) \leq 1.33$

S = tolerable settlement in mm.

- Empirical correlations for $q_{net(all)}$ with CPT(q_c) based on 1 inch settlement (Meyerhof)

For B $\leq$ 1.22m

$$q_{net(all)} = q_c / 15$$

For B > 1.22m,

$$q_{net(all)}(kN/m^2) = \frac{q_c}{25}\left(\frac{3.28B+1}{3.28B}\right)^2$$

in meters and kN/m^2.

- Notes

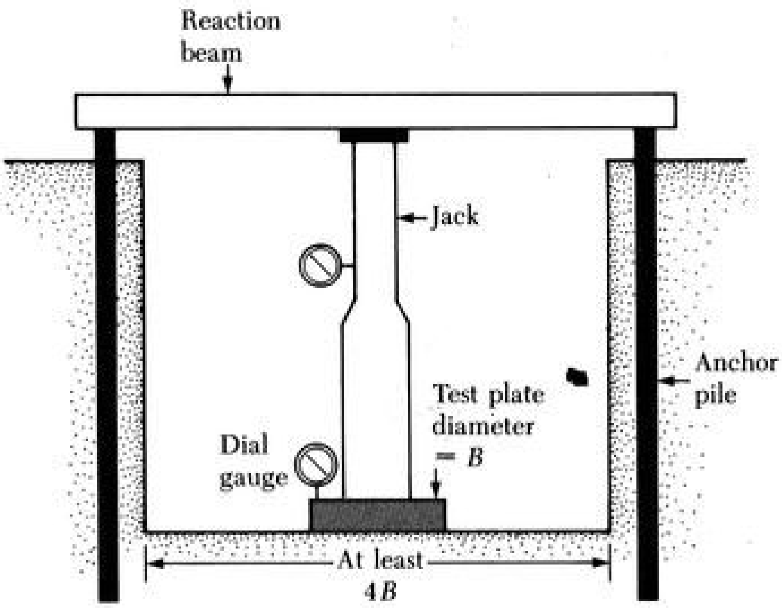
1)

2)

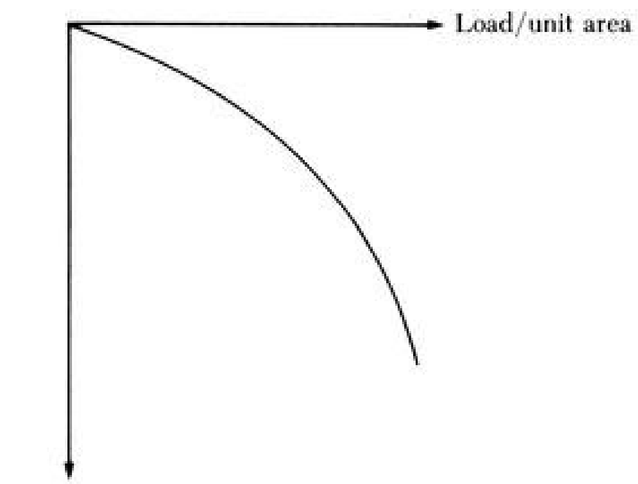
3)

12) Field Load Test (Plate Load Test)

- Test Method
  - Steel plate : 25mm thickness and 150mm~762mm diameter
    (or square plate with 305mm x 305mm)
  - A hole is excavated with a minimum diameter of 4B to a depth of D_f.
    (D_f : depth to the proposed foundation)



(a)



(b)

- For tests in clay,

$$q_{u(F)} = q_{u(p)}$$

where,
$q_{u(F)}$ = ultimate bearing capacity of the proposed foundation
$q_{u(p)}$ = ultimate bearing capacity of the test plate

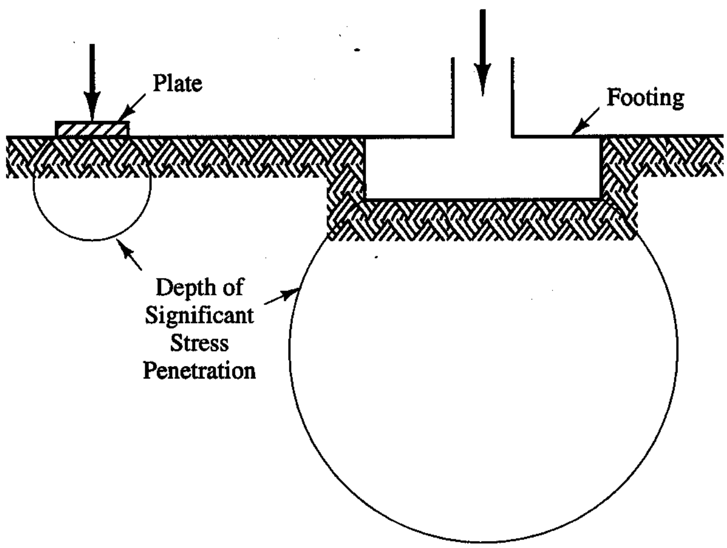
- For tests in sand,

$$q_{u(F)} = q_{u(p)} \frac{B_F}{B_p}$$

where,
B_F = width of the foundation
B_p = width of the test plate

- Major drawback of plate load test

Effect of footing size difference
⇒
⇒

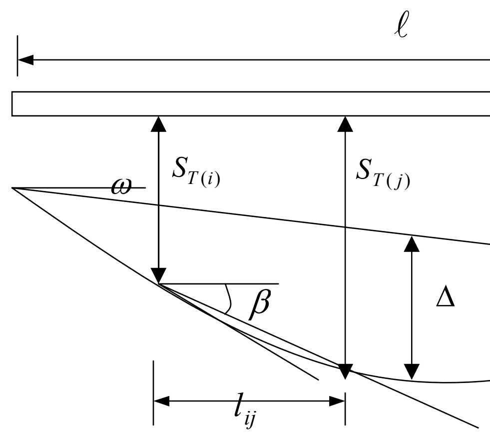


13) Tolerable Settlements

- Non-homogeneous subsoil conditions + wide variation of the load carried by various shallow foundations.

  ➡

  ➡



- <u>The parameters for definition of tolerable settlement</u>

  * S_T = Total vertical displacement at point i
  * ΔS_T = Differential settlement between i and j = $S_{T(j)} - S_{T(i)}$
  * Δ = Relative deflection
  * ω = Tilt
  * β = Angular distortion = $\Delta S_T / l_{ij}$
  * Δ / L = Deflection ratio

- Criteria for tolerable settlement are determined based on types and functions of structures.

- Skempton and McDonald

  - Maximum settlements, $S_{T(\max)}$

    In sand 32mm
    In clay 45mm

  - Maximum differential settlements, $\Delta S_{T(\max)}$

    Isolated foundations in sand 51mm
    Isolated foundations in clay 76mm
    Raft in sand 51~76mm
    Raft in clay 76~127mm

  - Maximum angular distortion 1/300

- Soviet Code of Practice

  - Allowable deflection ratio is a function of L/H, the ratio of the length to the height of a building.

Type of Building	L/H	Δ/L
Multistory building and civil dwelling	≤ 3	0.0003 (for sand) 0.0004 (for clay)
	≥ 5	0.0005 (for sand) 0.0007 (for clay)
One-story mills		0.001 (for sand and clay)

- Bjerrum

Category of potential damage	β_{max}
Safe limit for flexible brick wall (L/H > 4)	1/150
Danger of structural damage to most buildings	1/150
Cracking of panel and brick walls	1/150
Visible tilting of high rigid buildings	1/250
First cracking of panel walls	1/300
Safe limit for no cracking of building	1/500
Danger to frames with diagonals	1/600

- Allowable deflection ratio is a function of L/H, the ratio of the length to the height of a building.

- European Committee

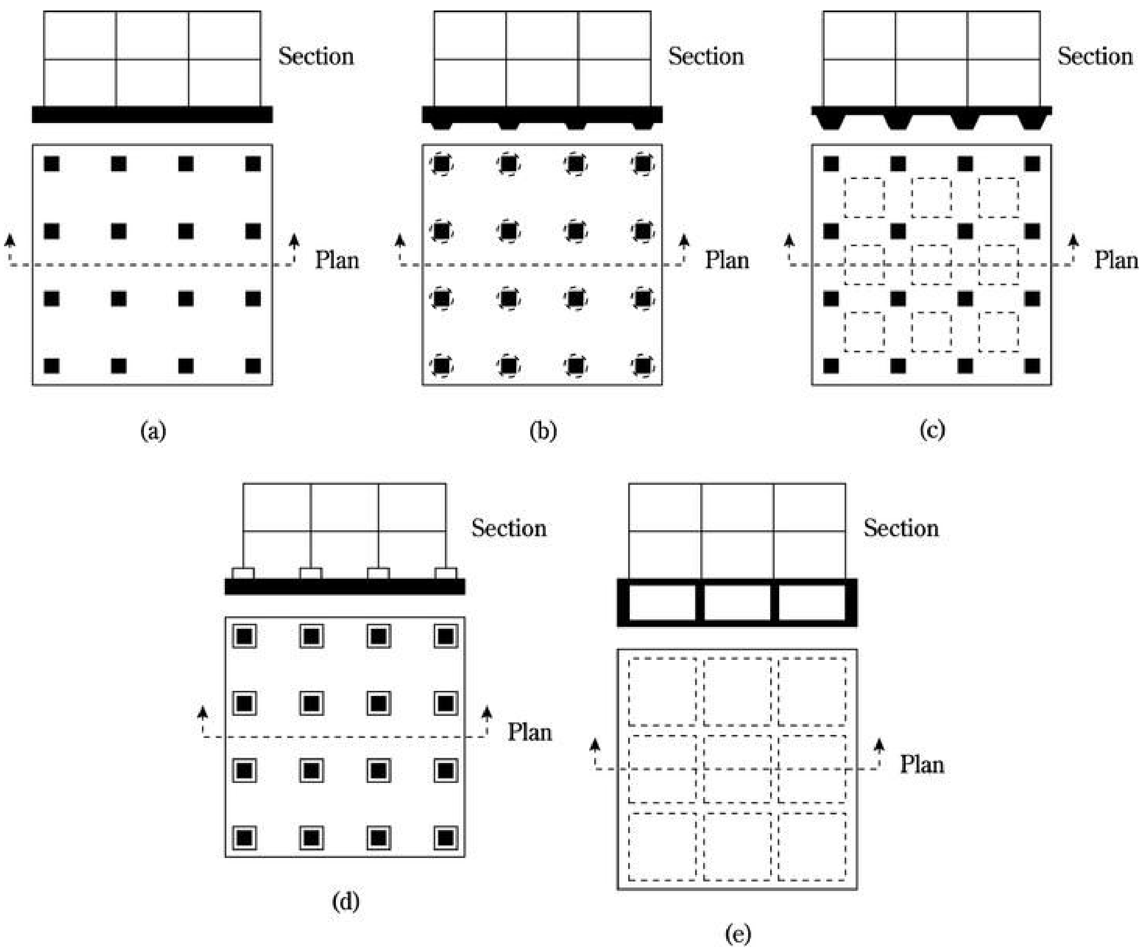
Item	Parameter	Magnitude	Comments
Limiting values	S_T	25mm	Isolated shallow foundation
for serviceability		50mm	Raft foundation
(European Committee	ΔS_T	5mm	Frames with rigid cladding
for Srandardization, 1994a)		10mm	Frames with flexible cladding
		20mm	Open frames
	β	1/500	-
Maximum acceptable	S_T	50	Isolated shallow foundation
foundation movement	ΔS_T	20	Isolated shallow foundation
(European Committee	β	$\approx$ 1/500	-
for Srandardization, 1994b)			

14) Mat Foundations

- When individual footing plan areas cover 50~75% of the plan area of the structure, use a mat foundation.

- Types of mat foundations (based on rigidity of footing)

  a) Flat plate
  b) Flat plate thickened under columns
  c) Beams and slabs
  d) Flat plates with pedestals
  e) Slab with basement walls as a part of the mat



- Points for design

- <u>Bearing Capacity</u>

  - Bearing capacity can be obtained with the same formula for spread footing.

$$q_u = cN_c F_{cs} F_{cd} F_{ci} + qN_q F_{qs} F_{qd} F_{qi} + 1/2\gamma B F_{\gamma s} F_{\gamma d} F_{\gamma i}$$

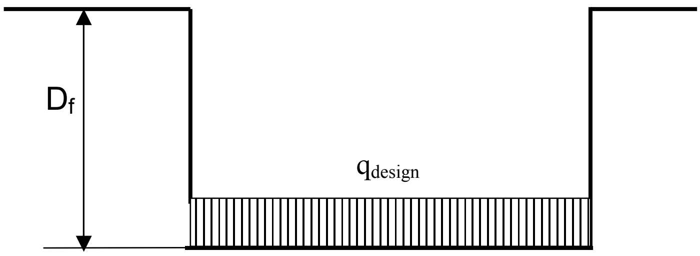
  - If embedded, the net applied load, q_n

$$q_n = q_{design} - \gamma D_f$$

  - So, for bearing capacity stability

$$q_{all(net)} = (q_u - \gamma D_f)/FS \geq q_n$$

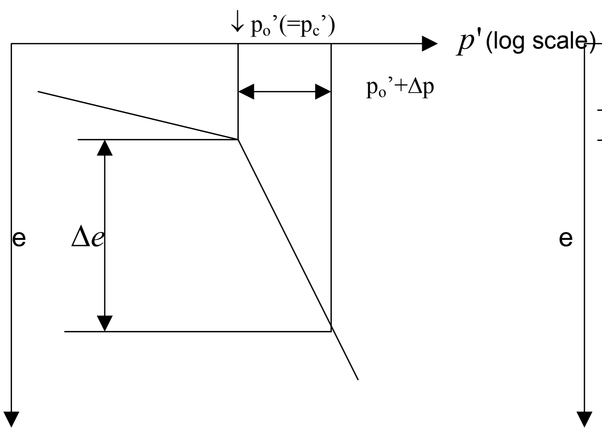
(or $q_{all} = q_u / FS \geq q_{design}$)



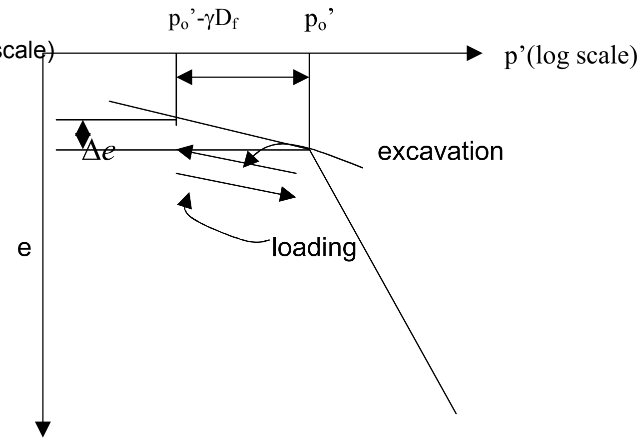
  - Compensated foundation ;

No increase of the net pressure on soil below a mat foundation. ➨ $q_n = q_{design} - \gamma D_f = 0$, $D_f = q_{design}/\gamma$ ➨ Effective on reducing the settlements.

For NC clays



(a) No compensating



(b) Fully compensating

- Allowable bearing pressure using SPT tests (Bowles);

  From Eq.(5.79),

$$q_{all(net)}(KN/m^2) = 11.98N_{cor}\left(\frac{3.28B+1}{3.28B}\right)^2(1+0.33(D_f/B))(S_e/25.4)$$

where S_e =settlement, in mm.

For mat foundation, B is very large. => $3.28B+1 \approx 3.28B$

$$q_{all(net)}(KN/m^2) = 11.98N_{cor}(1+0.33(D_f/B))(S_e/25.4) \le 15.93N_{cor}(S_e/25.4)$$

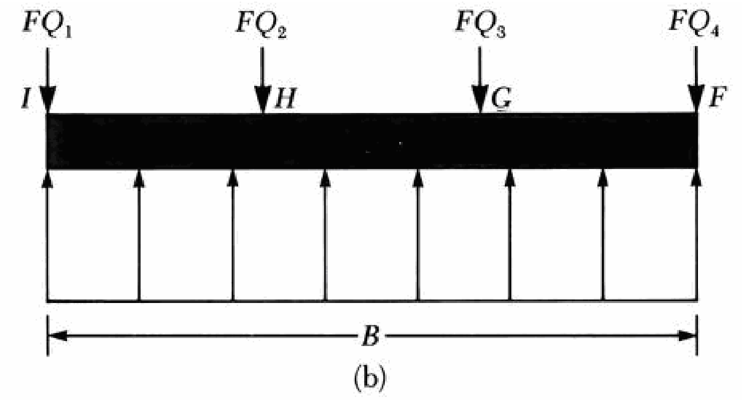
- Based on 2 inch total settlement and conservatively assuming that $F_d(=1+0.33(D_f/B))=1$

$$q_{all(net)}(kN/m^2) = 23.96N_{cor}$$

- <u>Structural Design of Mat foundations</u>

i) Conventional rigid method

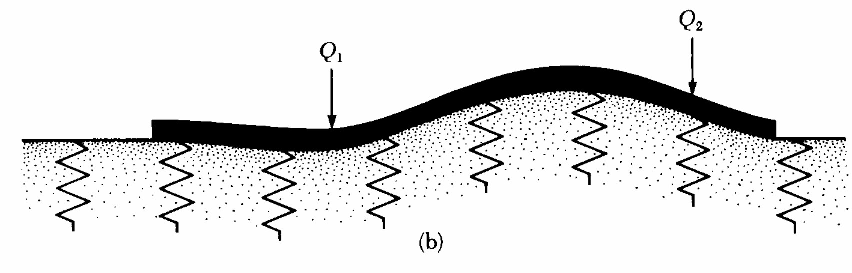
- Bearing pressure : Uniform distribution for no eccentric loading
  Varying linearly across the footing for eccentric loading.
- Mat : Infinitely rigid and an inverted simply loaded two-way slab.



(Fig. 6.11b)

ii) Beam(slab) on elastic foundation (Winkler Foundation)

- Define the relationship between bearing pressure and settlement using the modulus of subgrade reaction, $k(= q/\delta, kN/m^2/m)$
- Then model the foundation soil as a bed of springs with a stiffness k_S.



(Fig. 6.12b)

* Case study

1. Bearing capacity failure (Coduto, p181, Collapse of the Fargo Grain Elevator)
2. Field settlement observations for foundations (p.304, 5.5)

Collapse of the Fargo Grain Elevator

One of the most dramatic bearing capacity failures was the Fargo Grain Elevator collapse of 1955. This grain elevator, shown in Figure 6.12, was built near Fargo, North Dakota, in 1954. It was a reinforced concrete structure composed of 20 cylindrical bins and other appurtenant structures, all supported on a 52 ft (15.8 m) wide, 218 ft (66.4 m) long, 2 ft 4 in (0.71 m) thick mat foundation.

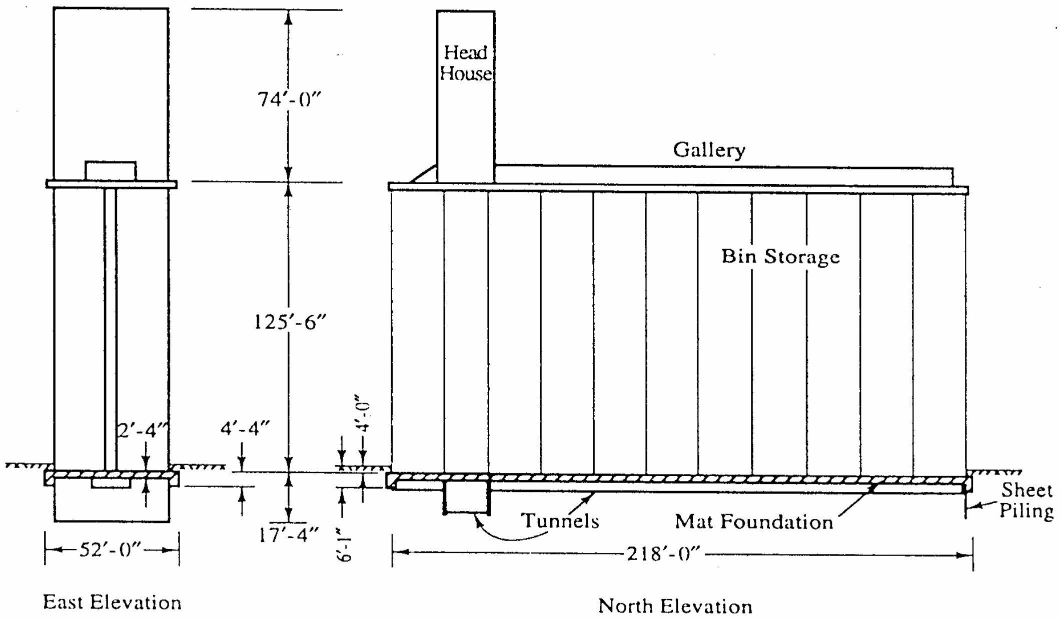


Figure 6.12 Elevation views of the elevator (Nordlund and Deere, 1970; Reprinted by permission of ASCE).

The average net bearing pressure, q', due to the weight of the empty structure was 1590 lb/ft^2 (76.1 kPa). When the bins began to be filled with grain in April 1955, q' began to rise, as shown in Figure 6.13. In this type of structure, the live load (i.e. the grain) is much larger than the dead load; so by mid-June, the average net bearing pressure had tripled and reached 4750 lb/ft^2 (227 kPa). Unfortunately, as the bearing pressure rose, the elevator began to settle at an accelerating rate, as shown in Figure 6.14.

Early on the morning of June 12, 1955, the elevator collapsed and was completely destroyed. This failure was accompanied by the formation of a 6 ft (2 m) bulge, as shown in Figure 6.15.

No geotechnical investigation had been performed prior to the construction of the elevator, but Nordlund and Deere (1970) conducted an extensive after-the-fact investigation. They found that the soils were primarily saturated clays with s_u=600-1000 lb/ft^2 (30-50 kPa). Bearing capacity analyses based on this data indicated a net ultimate bearing capacity of 4110 to 6520 lb/ft^2 (197-312 kPa) which compared well with the q' at failure of 4750 lb/ft^2 (average) and 5210 lb/ft^2 (maximum).

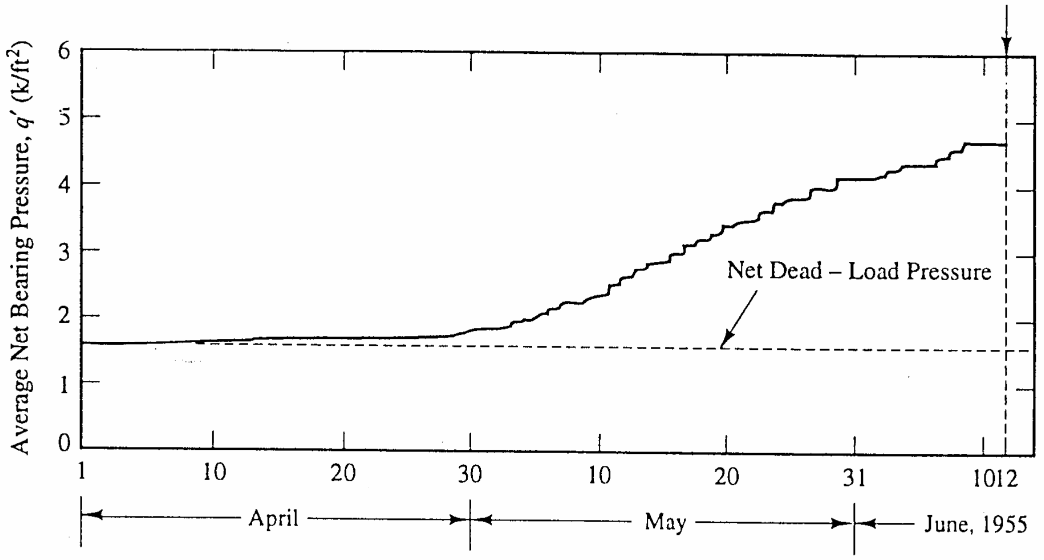


Figure 6.13 Rate of loading (Nordlund and Deere, 1970; Reprinted by permission of ASCE).

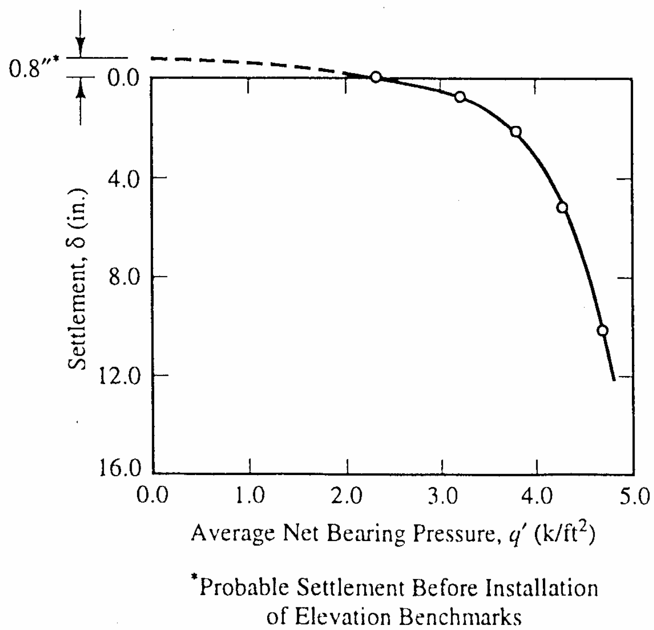


Figure 6.14 Settlement at centroid of mat (Nordlund and Deere, 1970; Reprinted by permission of ASCE).

In other words, this was a classic bearing capacity failure that easily could have been predicted in advance.

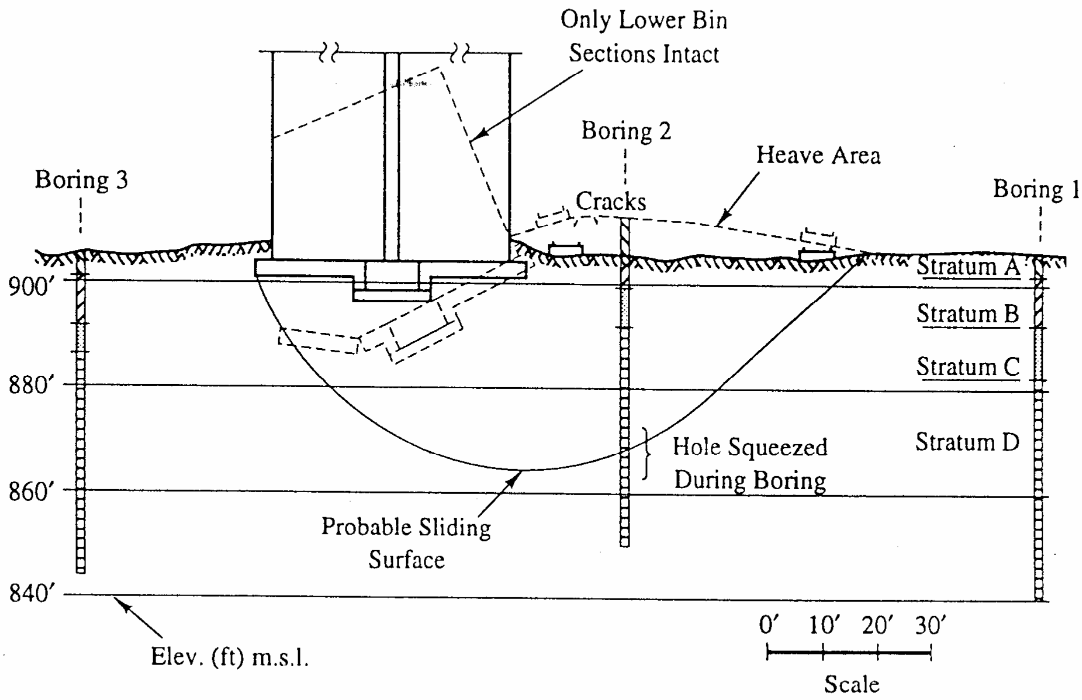


Figure 6.15 Cross-section of collapsed elevator (Nordlund and Deere, 1970; Reprinted by permission of ASCE)

Although bearing capacity failures of this size are unusual, this failure was not without precedent. A very similar failure occurred in 1913 at a grain elevator near Winnipeg, Manitoba, approximately 200 miles (320 km) north of Fargo (Peck and Bryant, 1953; White, 1953). This elevator rotated to an inclination of 27° from the vertical when the soil below experienced a bearing capacity failure at an average q' of 4680 lb/ft^2 (224 kPa). The soil profiles at the two sites are very similar, as are the average q' values at failure. This is a classic example of engineers failing to learn from the mistakes of others.