Response to 4a. Don Bartlett 1/19/82 J Kane 14/B3

A DISCUSSION ON THE EFFECTS OF PHASE II CONSTRUCTION ON THE AUXILIARY BUILDING FOUNDATION

This discussion presents reasons why Phase II construction will not be detrimental to the foundation support of the auxiliary building. Phase II is primarily the construction of several 3 ft. by 6 ft. hand dug piers and 7 ft. high by 6 ft. wide access drifts necessary for access to the pier locations. Phase II does not include any undermining or removal of the supporting soil directly beneath the auxiliary building. Although there is lateral excavation adjacent to the materials supporting the auxiliary building, and there are excavations for hand dug piers, as explained below, these excavations and the construction of the piers will not be detrimental to the auxiliary building foundation.

The first consideration must be the strength and rigidity of the auxiliary building structure. The massive east-west shear wall is capable of redistributing the building loads to the underlying soil if necessary. A preliminary finite element analysis of the structure indicates that approximately 7 ksi maximum increase in rebar stress will occur if a 20 ft. width of soil were removed under both the east and west ends of the electrical penetration wings. This is a design case far more severe than any condition that could exist in Phase II construction. Therefore, this acceptable increase in stress provides assurance that the Phase II construction will not be detrimental to the auxiliary building foundation. In the actual case, there will not be any soil removed from under the auxiliary building; only a minor redistribution of the soil pressure bulb will take place, as a result of the construction.

Construction procedures are an important consideration. For the access drift, the procedure will be to advance the excavation approximately four feet without lagging. The unlagged excavation can be expected to stand at greater than 3 vertical to 1 horizontal during this stage of construction. After the excavation has been extended, a steel support frame will be installed four feet beyond the last in-place frame. Lagging will be placed along the sides of the drift between these two frames. Previously excavated soil will then be packed behind the lagging to restore lateral support to the unexcavated soil.

The pits will be constructed by the "excavate a foot - lag a foot" method in the fill material. Immediately after the lagging is in place, it will be backpacked to return lateral support to the surrounding soil.

These construction procedures for the access drifts and the pits are by controlled hand methods. They are also very localized construction activities. Additionally, no two adjacent pits will be worked on at the same time.

From field experience and the references listed at the end of this discussion, approximate limits of significantly disturbed soil adjacent to drift excavation can be expected to resemble the shape shown in Figures A1 and B1. The maximum horizontal projection of these zones of influence is approximately one half the height of the excavation. These figures, drawn to scale, indicate that the expected zones of influence do not extend to the soil supporting the auxiliary building.

The effect that the excavation will have on the "bulb of pressure" beneath the auxiliary building must also be evaluated. The vertical pressure in the supporting soil reduces with depth. The pressure lines on Figures A2 and B2 represent the bulb of pressure corresponding to one-tenth of the contact pressure beneath the foundation of the auxiliary building. Thus, it is seen from Figures A2 and B2 that this one-tenth ratio line does not intersect the access drifts.

However, there is an overlap of the zone of influence of significantly disturbed soil from Figures A1 and B1 with the 0.1 pressure bulb . This overlap will cause a redistribution of pressure, but because it occurs in a zone of low pressure the effect on the auxiliary building will be insignificant.

In a similar manner, excavation for the pits will cause disturbance of the low stress regions of the pressure bulb created by the auxiliary building. Again, this is a minor redistribution having an insignificant effect.

A contingency plan for ground stabilization will be implemented if the soil is found to be instable, or if the instrumentation indicates movement of the auxiliary building. Drive horiz. bracing
Chemically grout

The above discussion clearly indicates that Phase II construction will not be detrimental to the auxiliary building.

REFERENCES

1. Foundation Design, Wayne C. Teng, page 125, 126.

2. NAVFAC DM-7, Department of Navy, Figure 13-8.

3. Rock Tunneling With Steel Supports, Proctor & White, page 62.

4. Cofferdams, White and Prentis, page 61.

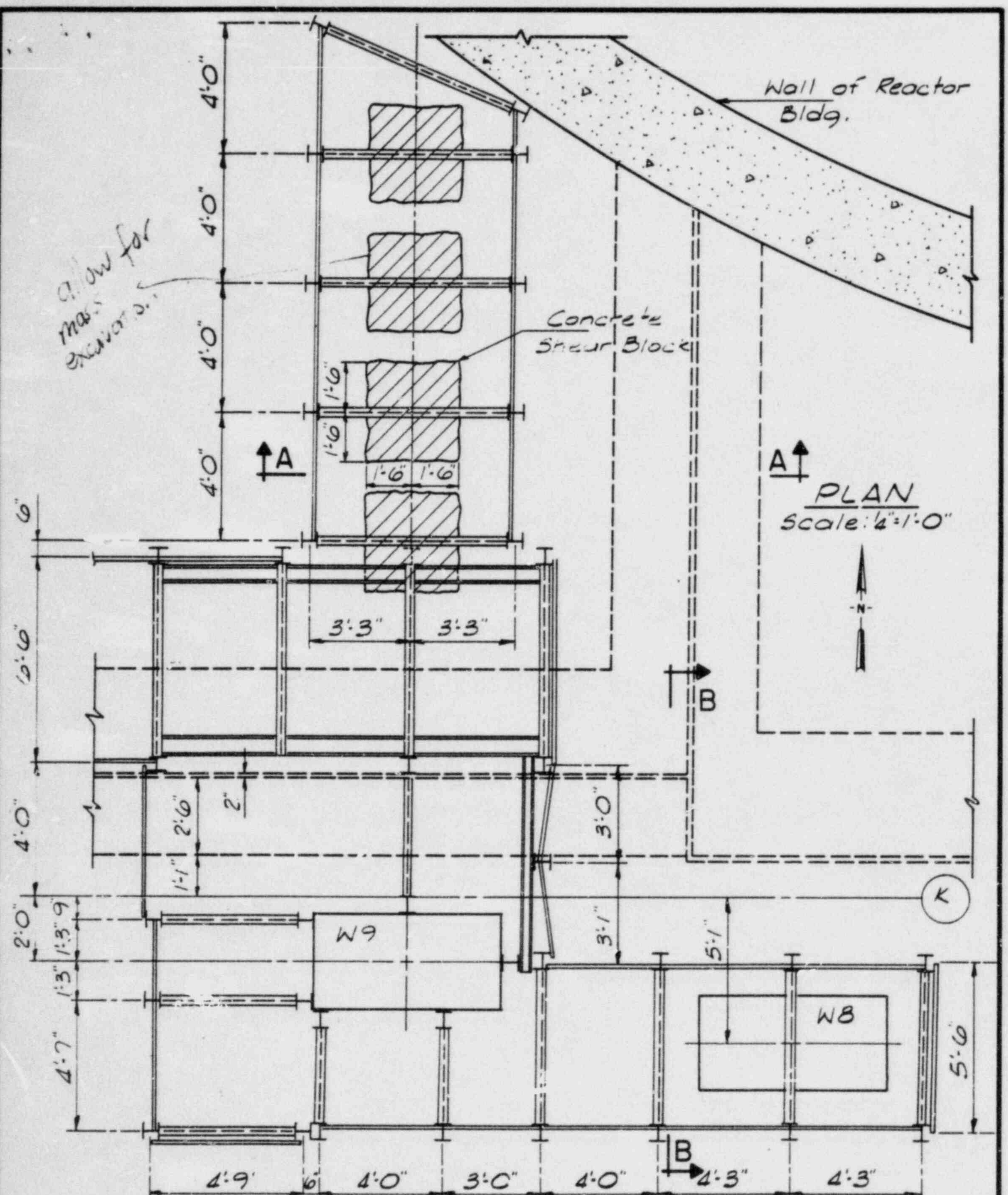

FIGURE A

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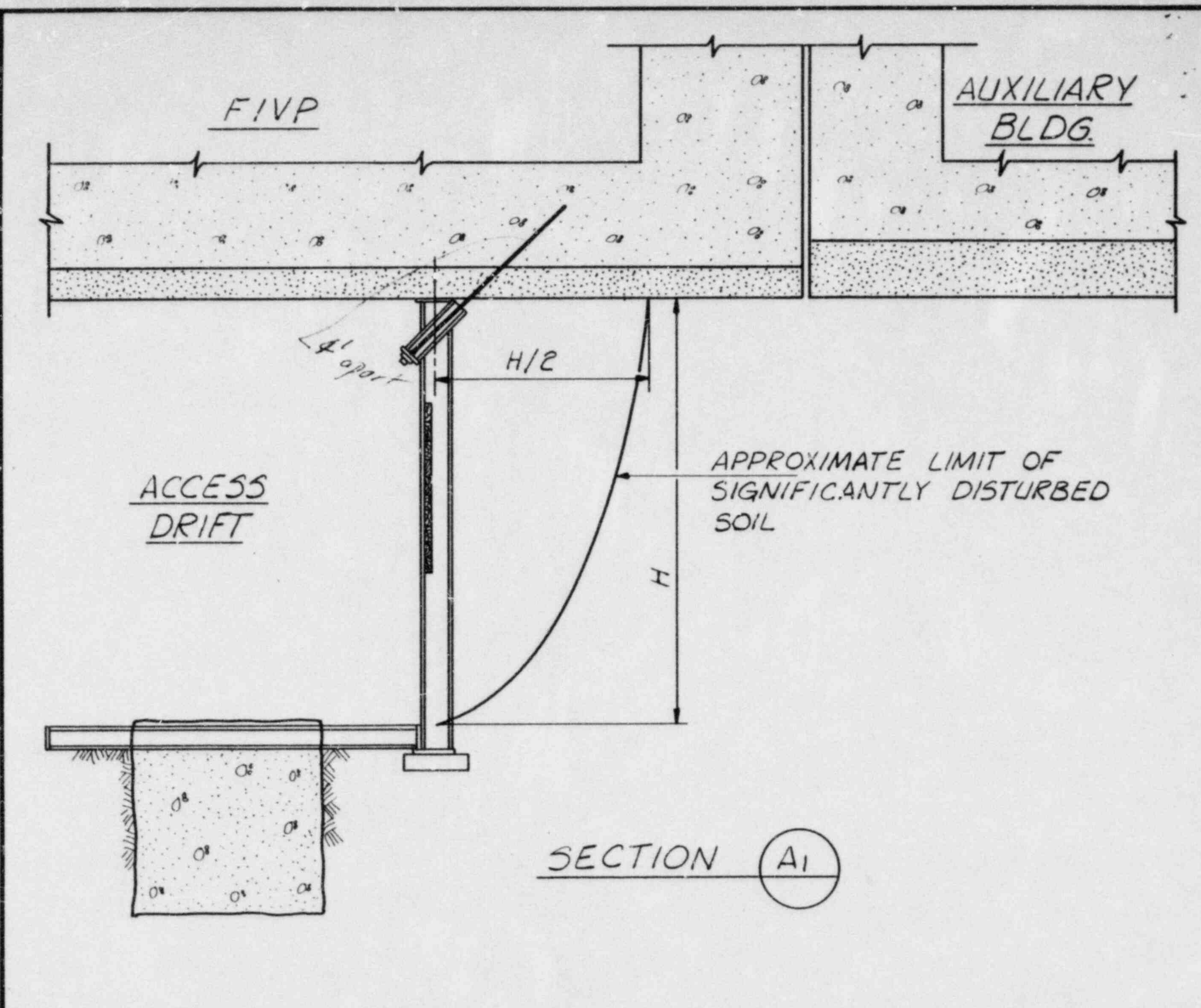

FIGURE A1

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FIGURE B1

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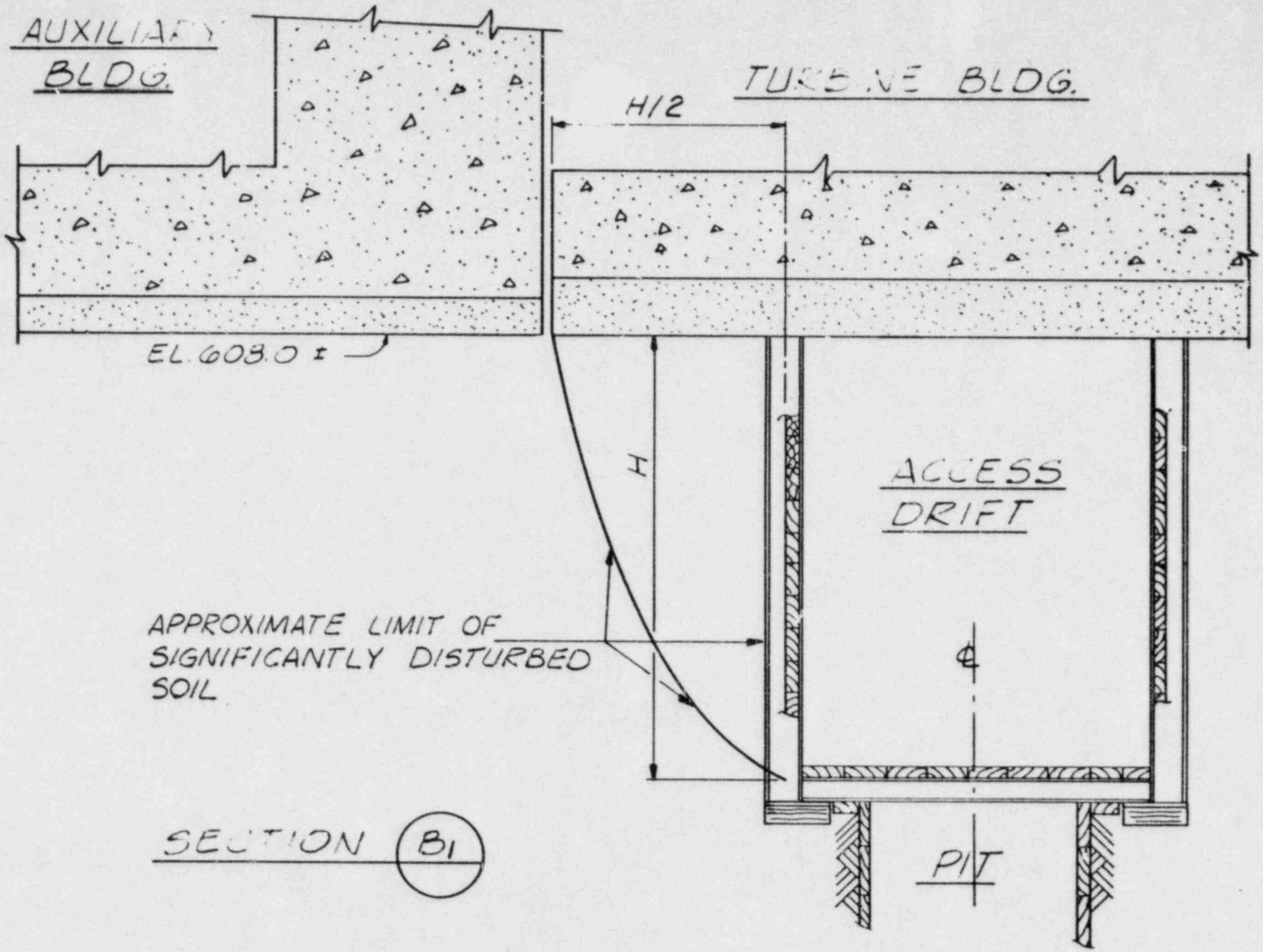
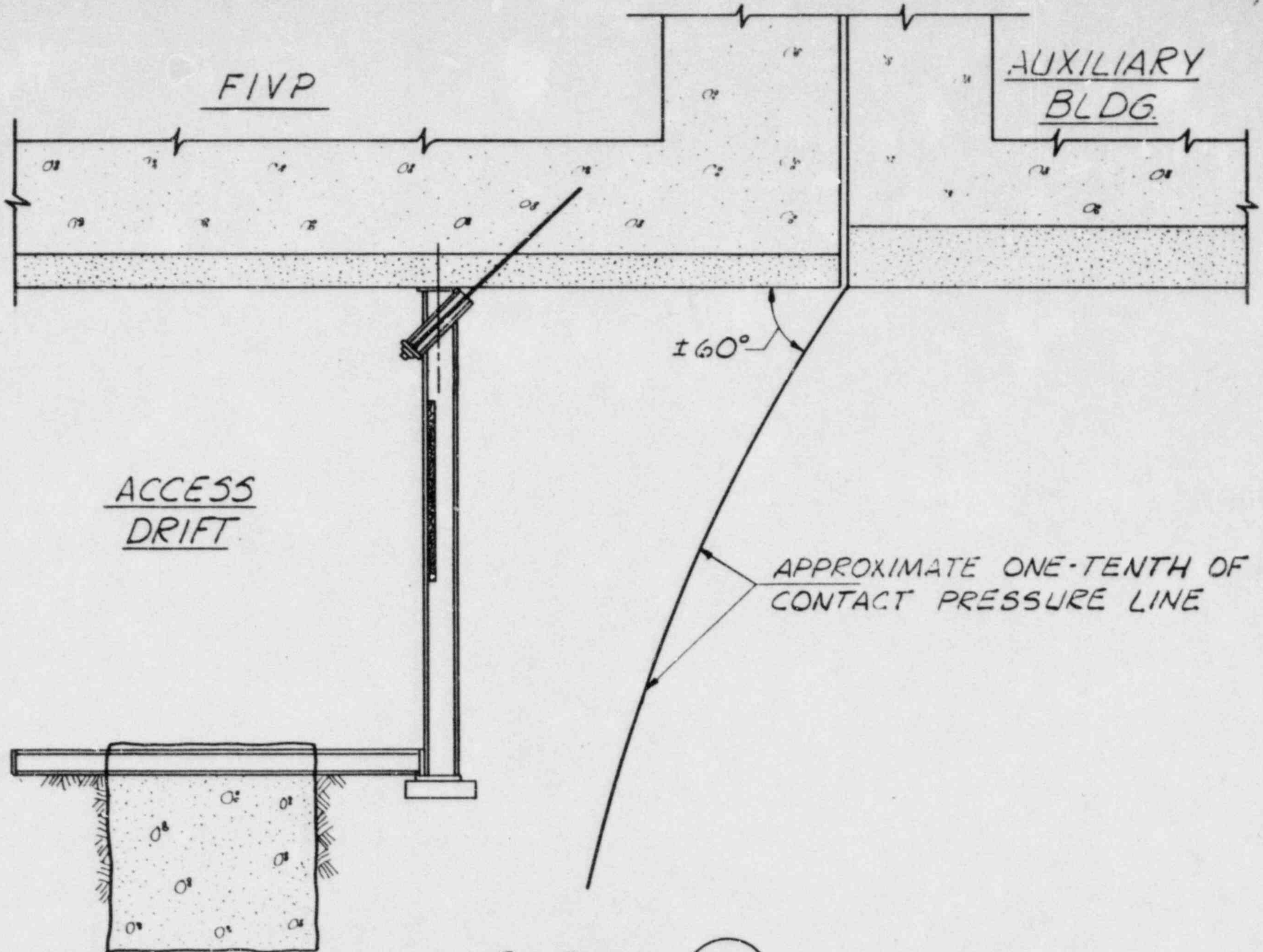


SECTION B1



SECTION A2

FIGURE A2

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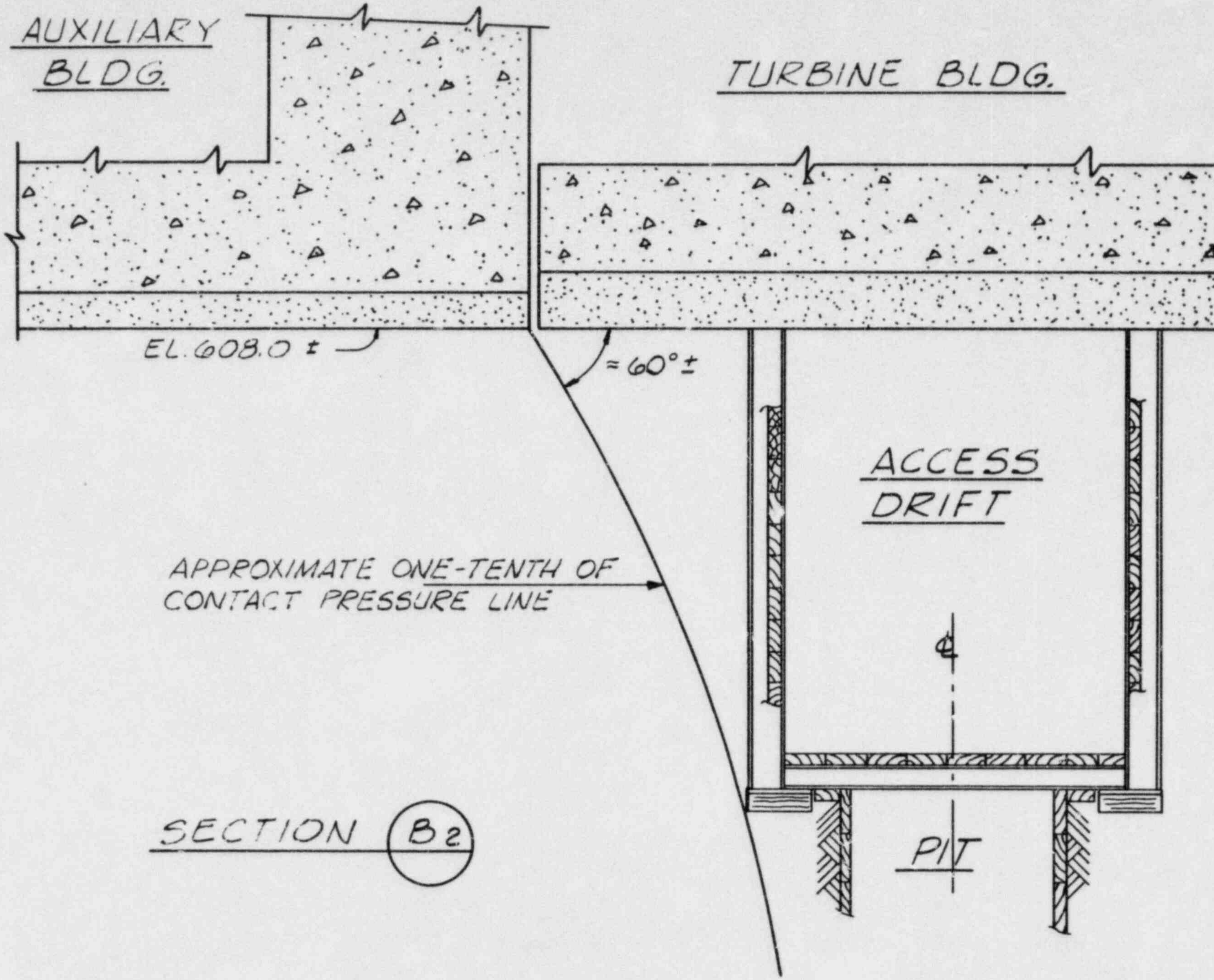

SECTION B2

FIGURE B2

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Let L_{l+d} = live load + dead load for the column which has the largest live load/dead load ratio;

L_s = service load for the same column;

= dead load + ½ live load for ordinary buildings;

q_a = allowable bearing pressure as determined by the principles discussed in Sec. 6-5;

q_d = design pressure for all footings except the one with largest live load/dead load ratio.

Then A = area of footing supporting the column with the largest live load/dead load ratio.

$= L_{l+d}/q_a$

$q_d = L_s/A$

$$\text{Area for other footings} = \frac{\text{Service load}}{q_d}$$

6-7 Stress on Lower Strata

1. For stability analysis of footings, the pressure under a footing may be assumed to spread out on a slope of 2 vertical to 1 horizontal. Thus, a load Q acting concentrically on a footing area of $B \times L$ is assumed to be distributed over an area of $(B + Z)(L + Z)$ at a depth Z below the footing, Fig. 6-8. If any stratum of soil is inadequate to sustain this spread-out pressure, the design bearing pressure should be reduced. However, for a two layer system of clays, the procedure described in Fig. 6-11 gives more reliable results.

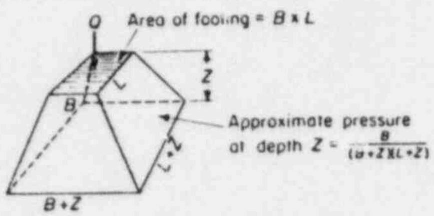


Fig. 6-8 Approximate distribution of vertical pressure under footing.

2. For settlement analysis, the approximation above may not be sufficient, and a more accurate approach based on elastic theory may be required. All elastic methods are developed from the Boussinesq's equation which deals with a single load acting on the surface of a half-space (infinitely large area and depth).

$$q = \frac{3Qz^3}{2\pi R^5} = \frac{3Q}{2\pi z^2}\cos^5\psi \tag{6-5}$$

where q = vertical stress at any given point;

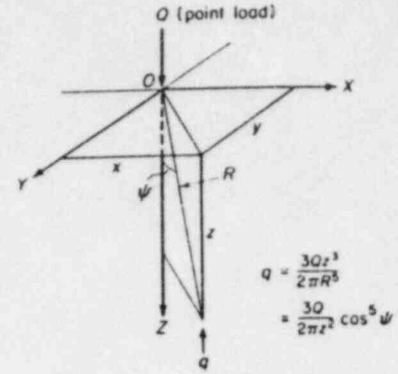


Fig. 6-9 Vertical stress due to a point load.

Q = surface load;

z = depth of the given point;

$r = \sqrt{x^2 + y^2 + z^2}$, see Fig. 6-9;

ψ = angle between line R and vertical.

Based on Boussinesq's equation, the vertical stresses under continuous, rectangular and circular footings have been computed. The results are shown in Fig. 6-10. In these figures the magnitude of vertical pressure at various points are given in terms of the bearing pressure q. For example the vertical pressure at any point along the line $0.2q$ is equal to 20

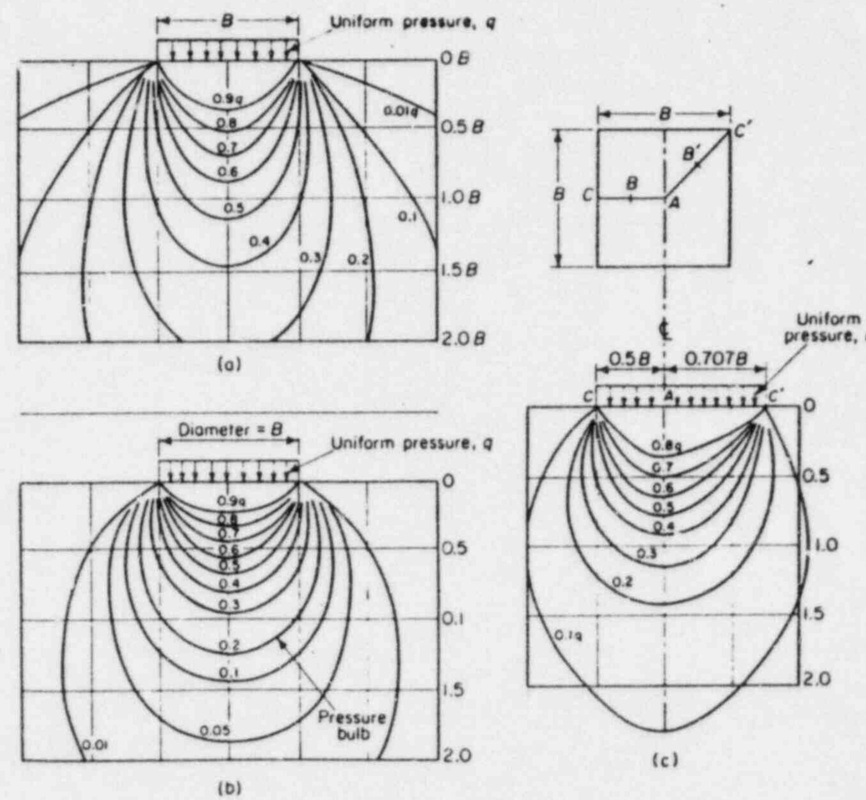
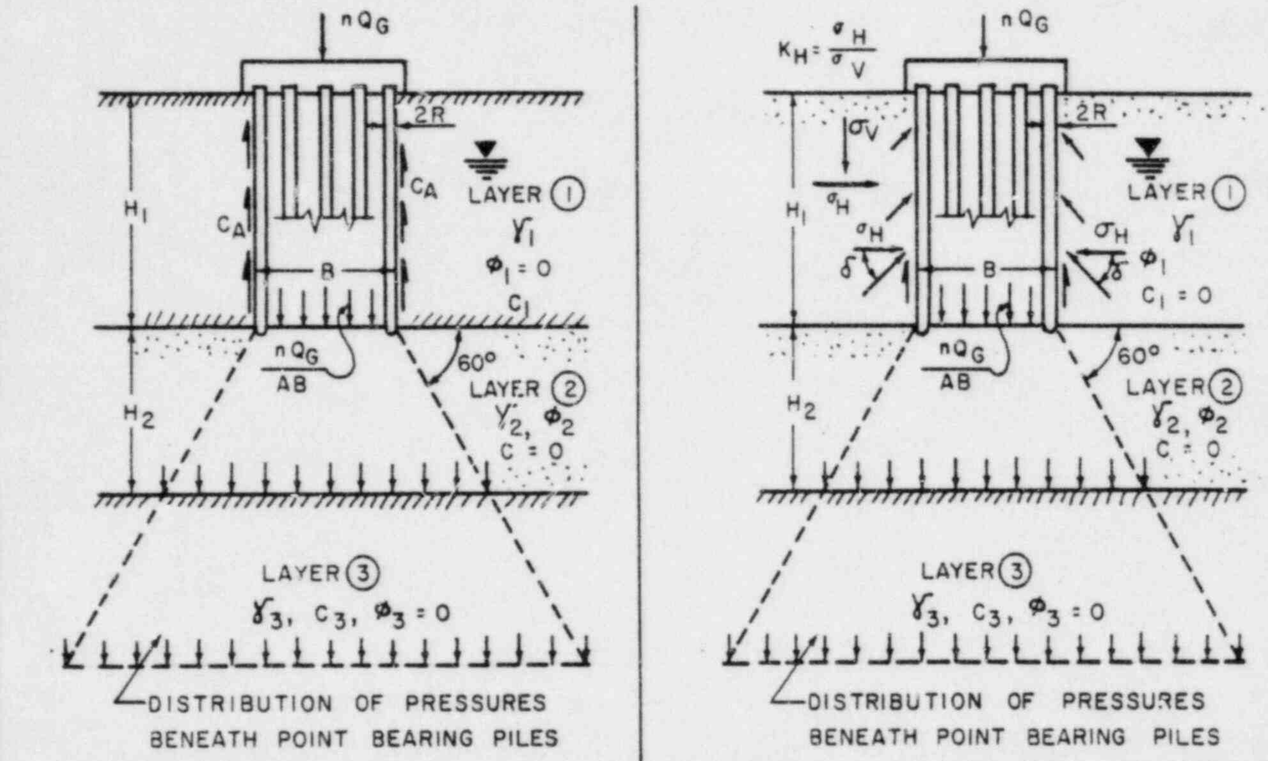


Fig. 6-10 Vertical stresses under footing: (a) under a continuous footing: (b) under a circular footing; (c) under a square footing.

OUTSIDE DIMENSIONS OF PILE GROUP IN PLAN = A × B, (B) IS SMALLER DIMENSION. PILES STOP IN TOP OF COARSE GRAINED LAYER (2). LAYER (2) IS UNDERLAIN BY COHESIVE STRATUM. LAYER (3). n = NUMBER OF PILES.



LAYER (1) IS COHESIVE ($\phi = 0$)	LAYER (1) IS COHESIONLESS ($C = 0$)
nQ_G = ULTIMATE LOAD CAPACITY OF GROUP Q_{ult} = ULTIMATE CAPACITY OF SINGLE PILE (WEIGHT OF PILES NEED NOT BE INCLUDED IN APPLIED LOAD). FAILURE IN LAYER (2) ($H_2 \geq B$) PILE SPACING $\leq$ 6R: $nQ_G = (\gamma_1 H_1 N_{q2} + 0.4 \gamma_2 B N_{\gamma 2}) A \times B + 2C_A (A+B) H_1 - AB\gamma_1 H_1$ PILE SPACING > 16R: $nQ_G = n(Q_{ult})$ $Q_{ult} = (\gamma_1 H_1 N_{q2} + 0.4 \gamma_2 B N_{\gamma 2}) \pi R^2 + 2C_A \pi R 4_1 - \pi R^2 \gamma_1 H_1$	FAILURE IN LAYER (2) ($H_2 \geq B$) IF GROUND WATER IS AT DEPTH GREATER THAN (B) BELOW TOP OF LAYER (2) : IF LAYER (1) IS ESSENTIALLY SIMILAR TO TO LAYER (2), OBTAIN nQ_G FROM FIG. 13-2. IF ϕ_1 DIFFERS GREATLY FROM ϕ_2: PILE SPACING < 6R: $nQ_G = (\gamma_1 H_1 N_{q2} + 0.4 \gamma_2 B N_{\gamma 2}) A \times B + (A+B) K_H \gamma_1 \tan\delta_1 H_1^2 - AB\gamma_1 H_1$ PILE SPACING > 16R: $nQ = n(Q_{ult})$ $Q_{ult} = (\gamma_1 H_1 N_{q2} + 0.4 \gamma_2 B N_{\gamma 2}) \pi R^2 + \pi R \gamma_1 K_H \tan\delta_1 H_1^2 - \pi R^2 \gamma_1 H_1$

FOR PILE SPACING BETWEEN 6R AND 16R, INTERPOLATE BETWEEN THE VALUES FOR 6R AND 16R.

FOR WATER NEAR TO THE GROUND SURFACE, SUBSTITUTE $\gamma_{1 sub}$ FOR γ_1 AND $\gamma_{2 sub}$ FOR γ_2 IN THE ABOVE FORMULAS. INTERPOLATE BETWEEN THESE LIMITS FOR INTERMEDIATE WATER LEVEL.

IN ANY CASE THE POSSIBILITY OF FAILURE IN CLAY LAYER (3) MUST BE INVESTIGATED. THIS IS PARTICULARLY IMPORTANT IF LAYER (2) IS THIN COMPARED TO DIMENSION (B). FAILURE OF LAYER (3) OCCURS IF LOAD DISTRIBUTED ON TOP OF LAYER (3) AS SHOWN EXCEEDS $1.3 C_3 N_c$.

FACTORS N_c N_γ & N_q OBTAINED FROM FIG. 11-1 FOR ALL CONDITIONS EXCEPT FOR COHESIONLESS SOILS WHEN LAYER (1) IS SIMILAR TO LAYER (2), IN THIS CASE USE N_c, N_γ AND N_q FROM FIG. 13-2.

FIGURE 13-8
Ultimate Load Capacity of Pile Groups in Layered Subsoils

The rock load H_p is represented in *Fig. 27* by the rectangle e f f_1 e_1. The balance of the weight of the overburden is carried by the ground arch. The weight of the middle part c d d_1 c_1 is transferred by the ribs of the tunnel support to the floor of the tunnel. The weight of the outer part acts as a surcharge on the top of the wedge-shaped bodies which tend to slide into the tunnel and increase the horizontal pressure exerted by these bodies.

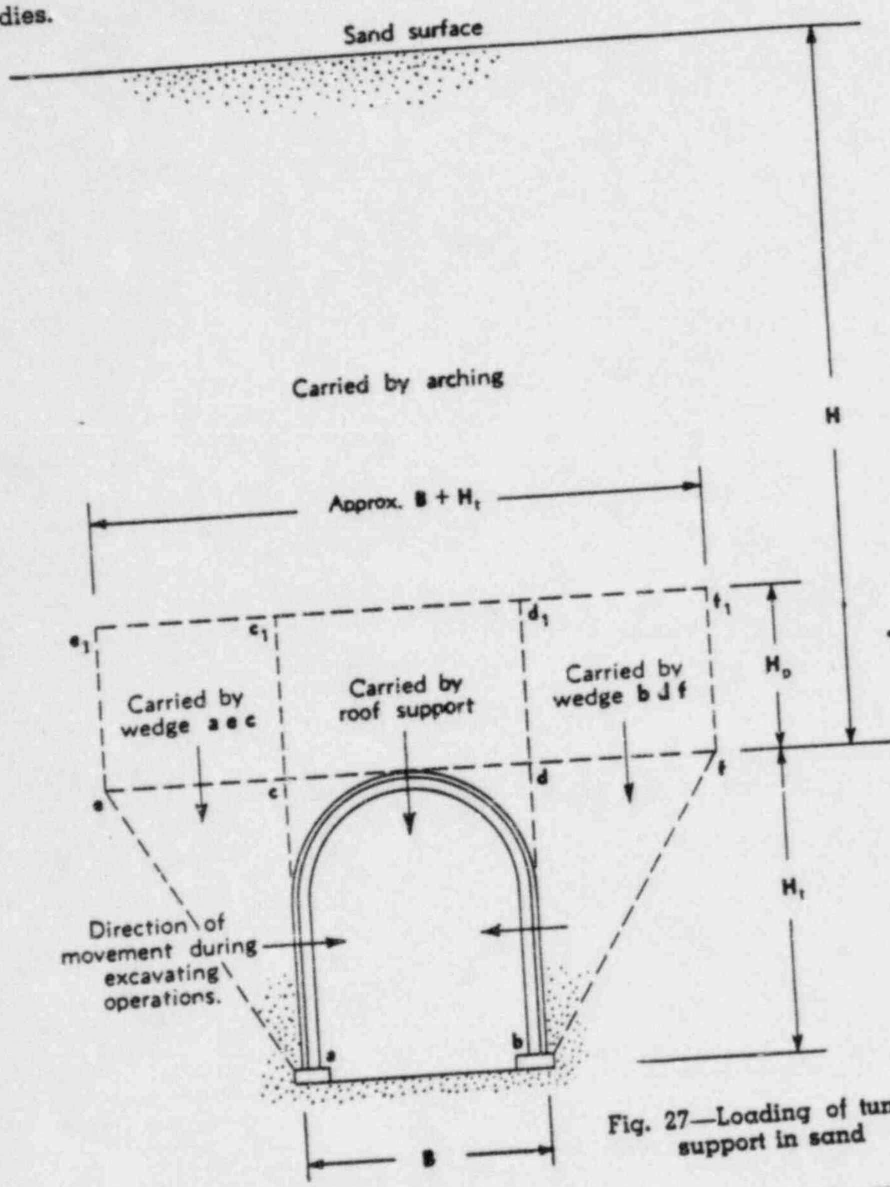


Fig. 27—Loading of tunnel support in sand

The rock load H_p is determined by eq. (2). According to the text accompanying this equation, the value of the constant C depends on the degree of compactness of the materials in which the tunnel is located and on the distance d through which the crown of the ground arch yielded before the support was installed. The distance d is not known and it can hardly be determined by practicable means. At a given width B of the tunnel it depends to a large extent on the skill of the miners and on the care with which the tunnel support is backpacked. The following numerical values are exclusively based on the results of the model tests with dry sand. Nevertheless it is

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$$P = \frac{H \times \frac{1}{2}H}{2} \times \frac{w}{2} = \frac{1}{8}wH^2$$

Comparing the above to the liquid pressure of a material of the same unit weight, we get a ratio of 0.25, as liquid pressure would be $\frac{1}{2}wH^2$. This ratio is called the coefficient K and was intro-

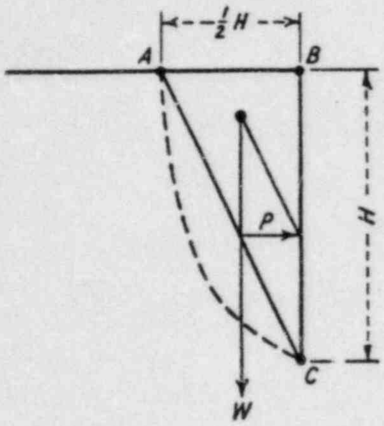

FIGURE 63. APPROXIMATE BREAK IN A BANK, SIMPLIFIED FOR COMPUTATION

duced by Terzaghi.[1] It is an aid to rough computations of earth pressures, but in many respects is misleading, as the distribution of pressure along the face of a solid may be entirely different from that produced by a liquid. It will be noted from Figure 63 that

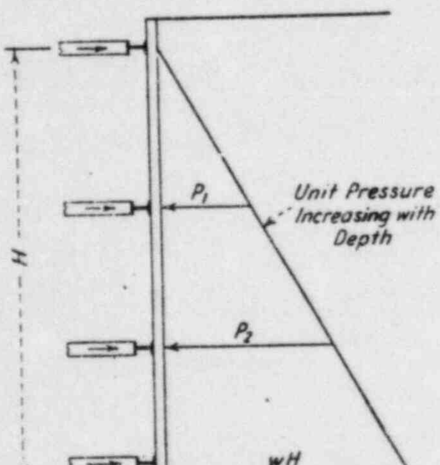

FIGURE 64. LIQUID PRESSURE ON A WALL

Total pressure (P) for unit width: $P = \frac{1}{2}wH^2$.

[1] *Soil Mechanics in Engineering Practice* by Karl Terzaghi and Ralph Peck. John Wiley & Sons, Inc., 1948, p. 353.