DESIGN OF UNDER~~GOUND~~ GROUND STRUCTURES FOR ATOMIC BLAST LOAD

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The advance in the design of intercontinental missiles (ICM) capable of carrying destructive atomic warheads to any part of the earth forces us to design shelter structures for military and civil defense purposes. In order to design efficiently for partial or complete protection, we must know first the destructive effects of atomic explosion, because knowing them, counter-measurements can be taken against the various effects.

The following brief description of atomic explosions is based on the recently released document of the U. S. Department of Defense and Atomic Energy Commission entitled "The Effects of Nuclear Weapons" by Dr. S. Glasstone. The problem of effects of nuclear burst is a complex one. Here, it will be merely attempted to present a general survey of the nuclear blast effects. The above mentioned non-classified government publication contains a considerable amount of detailed information. Consequently, in case of actual design, it is strongly recommended to use it.

The liberation of large nuclear energy, in case of atomic explosion, is accompanied by considerable increase in temperature of the ambient air or ground. Temperatures in the center of explosion are probably several millions of degrees -- about the same temperatures which exist on the surface of the sun where continuous nuclear explosions also take place. The gasses at very high temperature and pressure move out rapidly, pushing away air and earth with great force creating "blast effects" in the form of blast waves.

The high pressure wave moves very rapidly away from the center of explosion which is called, in the case of surface explosion, "Ground Zero." The front of the blast wave is called "Shock Front." Assuming an air-blast when the blast wave strikes the surface of the earth it is reflected back. The fusion of reflected waves and new blast waves is called "Mach Effect," Fig. 1.

The most critical case for protection purposes is the surface explosion, Fig. 2, because it creates a crater and the extreme blast pressure on the ground is the largest since incident and reflected waves coincide. As we will see later on, the so-called "Radioactive Fallout" is the most critical in case of surface blast also. Consequently, the structural designer would design for surface burst condition.

Since atomic bombs are still in evolution, we can say only at the present time that blast loads created on the surface exceeds 100 $\#/in^2$ without giving any upper limit. The blast pressure acting on the ground surface consists of pseudo-static pressure (P_s) created by the shock front and of dynamic pressures resulting from mass flow of air behind the shock wave $P_d = \frac{1}{2} \rho \mu^2$, where ρ = air density and μ = velocity of air particles.

Fig. 3 after Capt. F. E. Anderson shows the relation between pressure and distance from ground zero for 1 kiloton (KT). One kiloton bomb has the equivalent explosion energy of 100,000 ton conventional TNT explosives. (TNT = trinitrotolueno = nitrocompound containing NO_3). This table can be used for larger explosion energies multiplying the values obtained by the cube root of the ratios of explosion energies, (see Fig. 3). Pressures for a 20 MT (Megaton = millionton TNT) bomb at

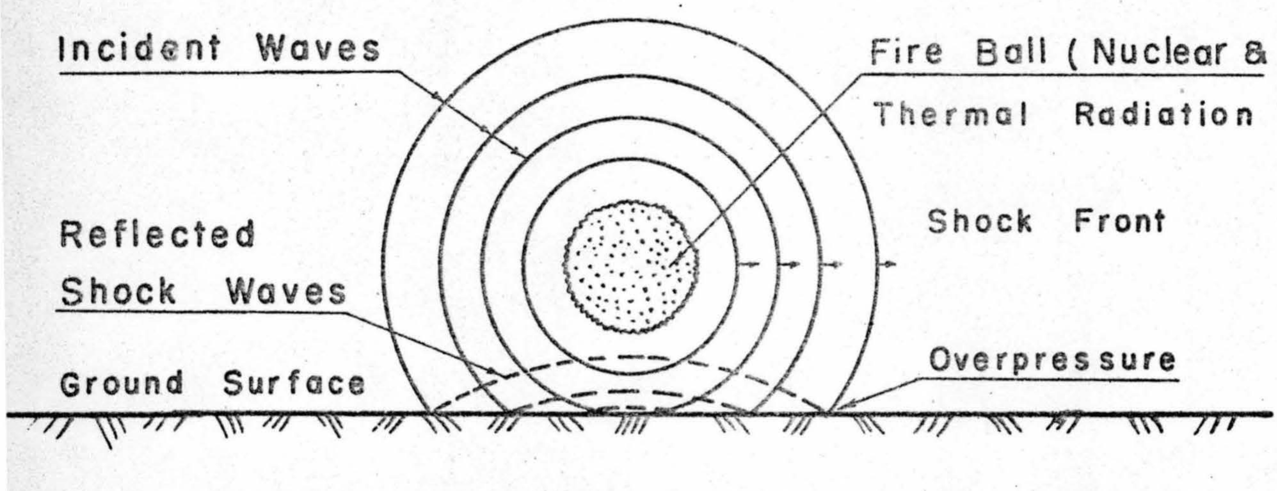


FIG. I BLAST WAVES FROM AIR BURST

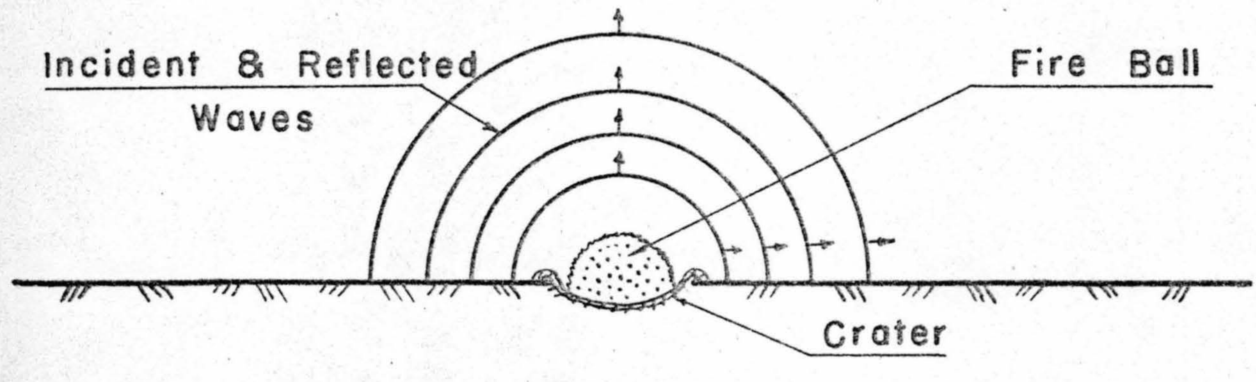


FIG. 2 BLAST WAVES FROM SURFACE BURST

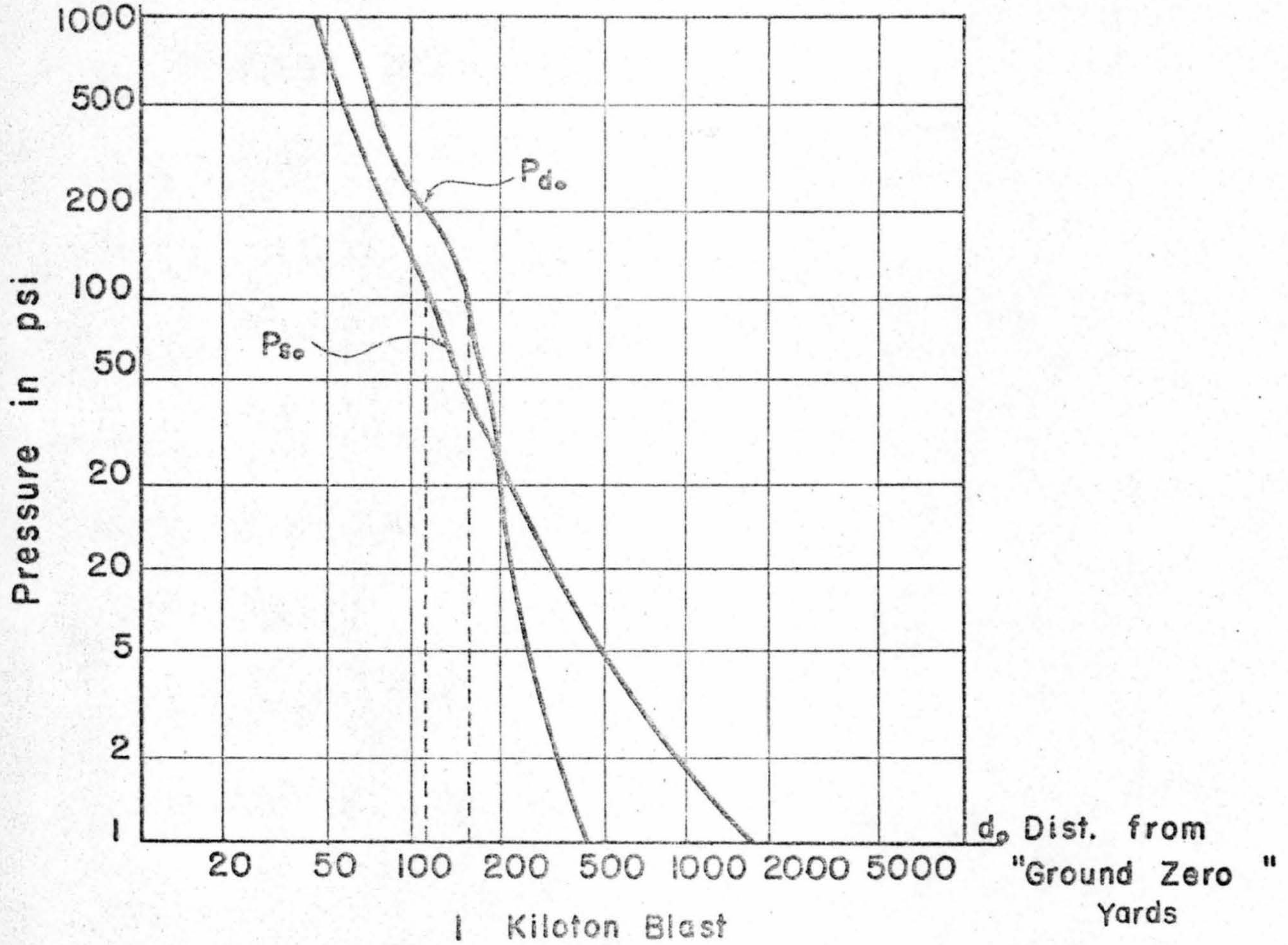


FIG. 3 RELATION BETWEEN PRESSURE & DISTANCE.

NOTE

For yields other than 1 KT the preasure & distance can be scaled as follows :

$$P_s + P_d = (P_{so} + P_{do}) \sqrt[3]{C}$$

$$d = d_o \sqrt[3]{C}$$

$$w = w_o \ C = \text{weapon yield energy}$$

Example : d required

$$P_{s+d} = 100 \text{ psi} \quad w = 20 \text{ MT}$$

$$C = \frac{20{,}000{,}000}{1000} = 20\,000$$

$$d = d_o \sqrt[3]{C}$$

$$= \frac{(110 + 160)}{2} \times 272 = 3725 \text{ yards}$$

ground "0" are extremely high which indicates that complete protection on the surface is almost impossible for blast pressure. It is mandatory to put air-raid shelters capable of withstanding nuclear blast underground which offers additional radiation, fallout, etc. protection.

At the explosion, considerable material is vaporized and sucked upward by ascending currents. Additional to that, soil has been displaced forming a crater. Fig. 4 shows a typical crater section. The dimensions of the crater are given again for 1 kiloton bomb assuming average dry soil, for other bomb energies again the given horizontal dimensions have to be multiplied by $\sqrt[3]{C}$ and the vertical by $\sqrt[4]{C}$ in the vicinity of the crater we can distinquish a so-called rupture zone where cracks of various sizes are caused by the explosion, and below that a plastic zone with permanent deformation of the soil. A complete protection at ground zero would require excessive depth. Generally, we are satisfied with a complete protection between 2000→5000 yards from ground zero using smaller units and taking chances by complete hit, since the probability of a complete hit is relatively small andgenerally does not justify the enormous costs involved by complete ground zero protection.

Before going into more detailed treatment of the structural design of underground structures we must mention the fundamental differences between nuclear and conventional explosions (TNT).

Nuclear explosion creates additional hazards such as:

1. Thermal radiation
2. Highly penetrating initial nuclear radiation
3. Residual nuclear radiation in form of atomic cloud (Fall-out)

The fire ball of the nuclear explosion resembles the sun in many

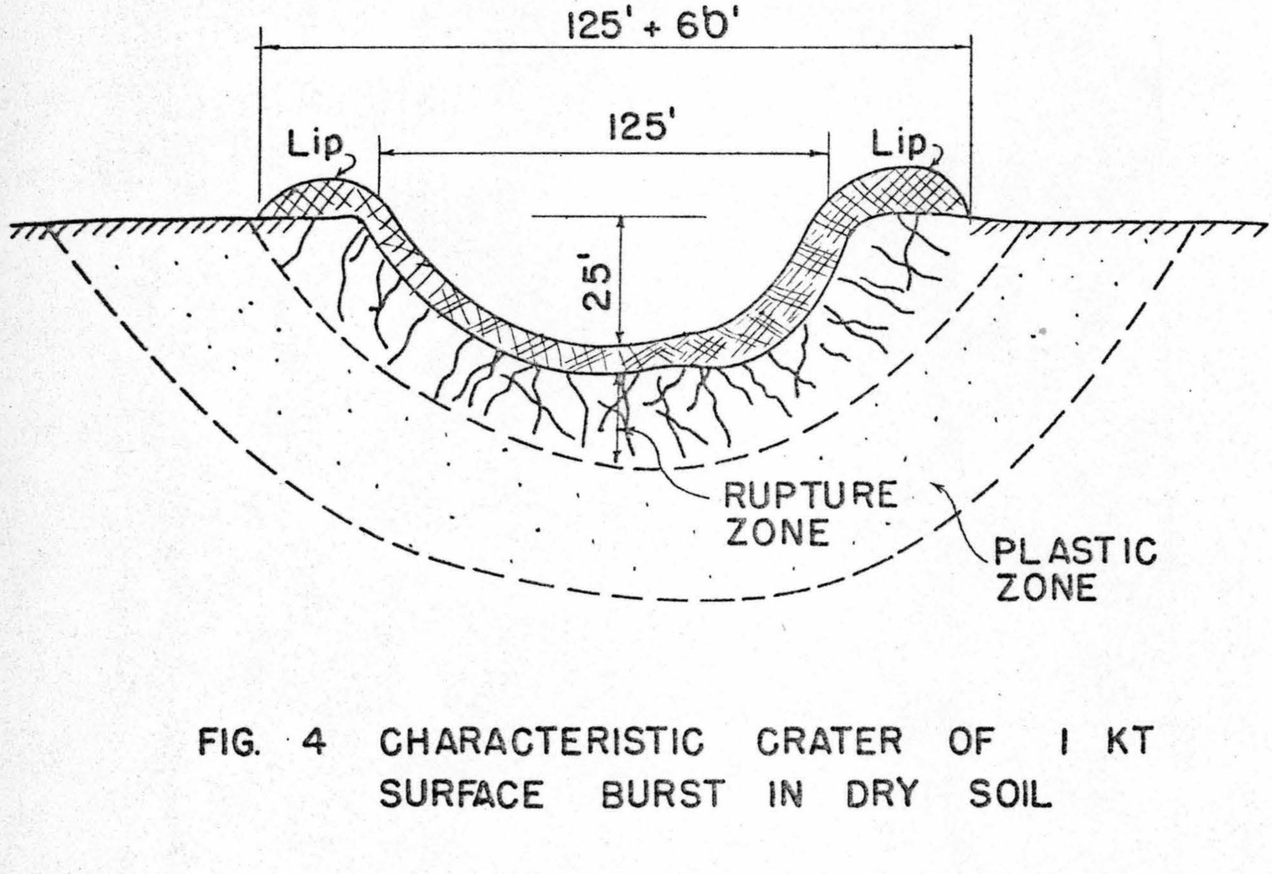


FIG. 4 CHARACTERISTIC CRATER OF 1 KT SURFACE BURST IN DRY SOIL

respects - radiating short wave length (visible), long wave ultraviolet and thermal radiations. Part of the thermal radiation on the structure may be reflected, other parts absorbed and the remainder, if any, will pass through causing burns and fires. Temperature falls rapidly with the distance from ground zero. Japanese experience during World War II (Hyroshima and Nagasaki) indicates that thermal radiation was 2900°F at 4000 ft away from ground zero. Shielding against thermal radiation is a fairly simple matter. An underground structure 10-20 ft below the surface has no additional fire protection and thermal radiation problems at the assumed 2000-5000 yards from ground zero.

Much more critical is the initial nuclear radiation. For example, at a distance of 1 mile from ground zero 1 Megaton (MT) bomb would kill 50% of the human beings even if they would be sheltered by 24" reinforced concrete structure. Nuclear radiation consists of γ rays (effects of which are similar to the effects of X-Rays), neutrons and β particles. Generally, γ rays like other rays are absorbed to some extent in the course of their passage through any material. The decrease in intensity of radiation depends largely on the density of material through which nuclear rays pass. Shielding can be achieved by proper amount of damp earth and concrete for γ ray as well as for neutron shielding. The calculation of the required thicknesses can be simplified by using diagrams shown in Fig. 5 and 6.

The effect of initial nuclear radiation due to γ rays and neutrons is expressed in REM = "Roentgen equivalent-units for man" since they cause similar injury to human beings. The maximum permissible radiation dosage is .03 REM per week = 15 REM per year. Figure 5 furnishes the total

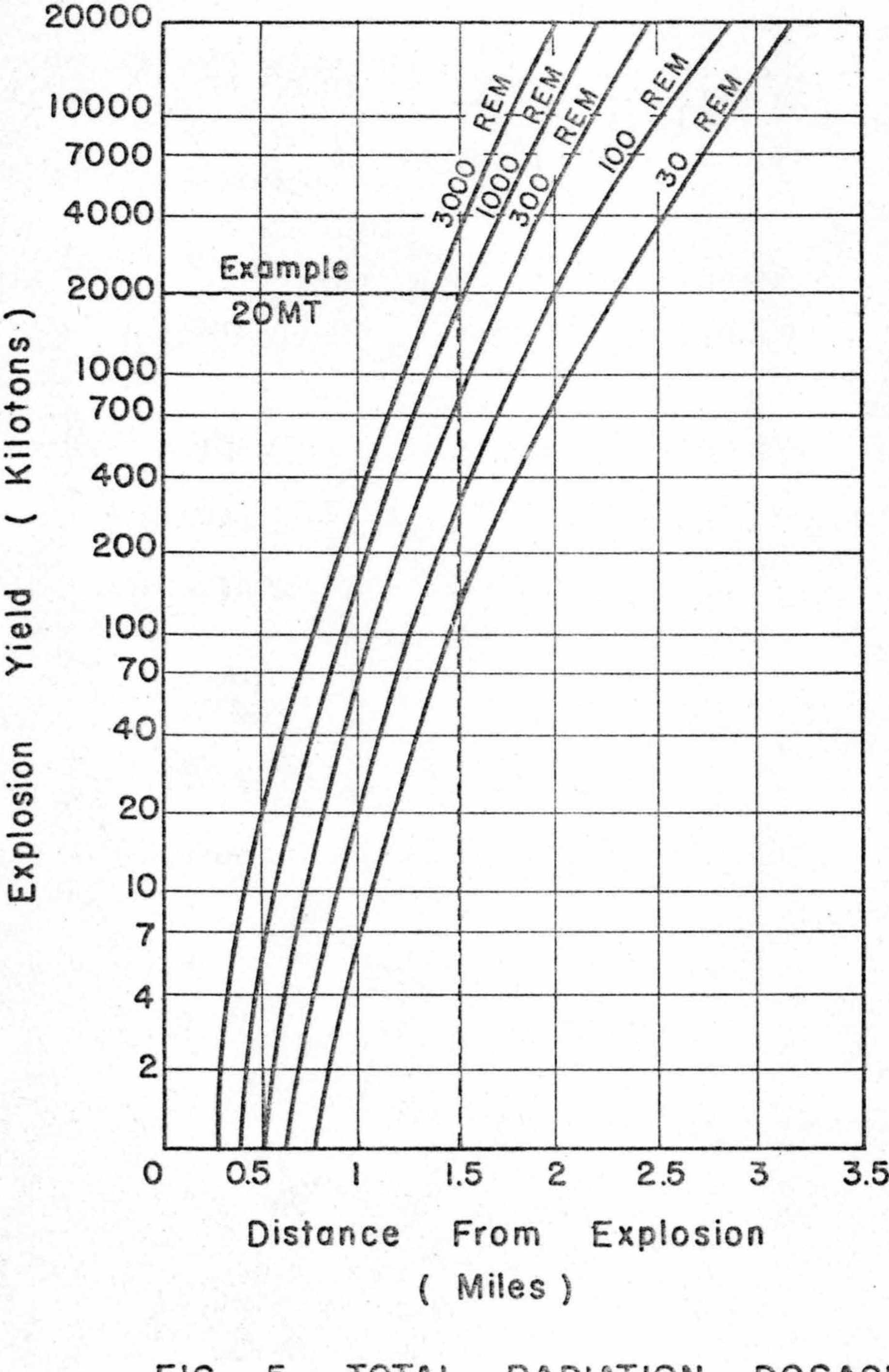


FIG. 5 TOTAL RADIATION DOSAGE

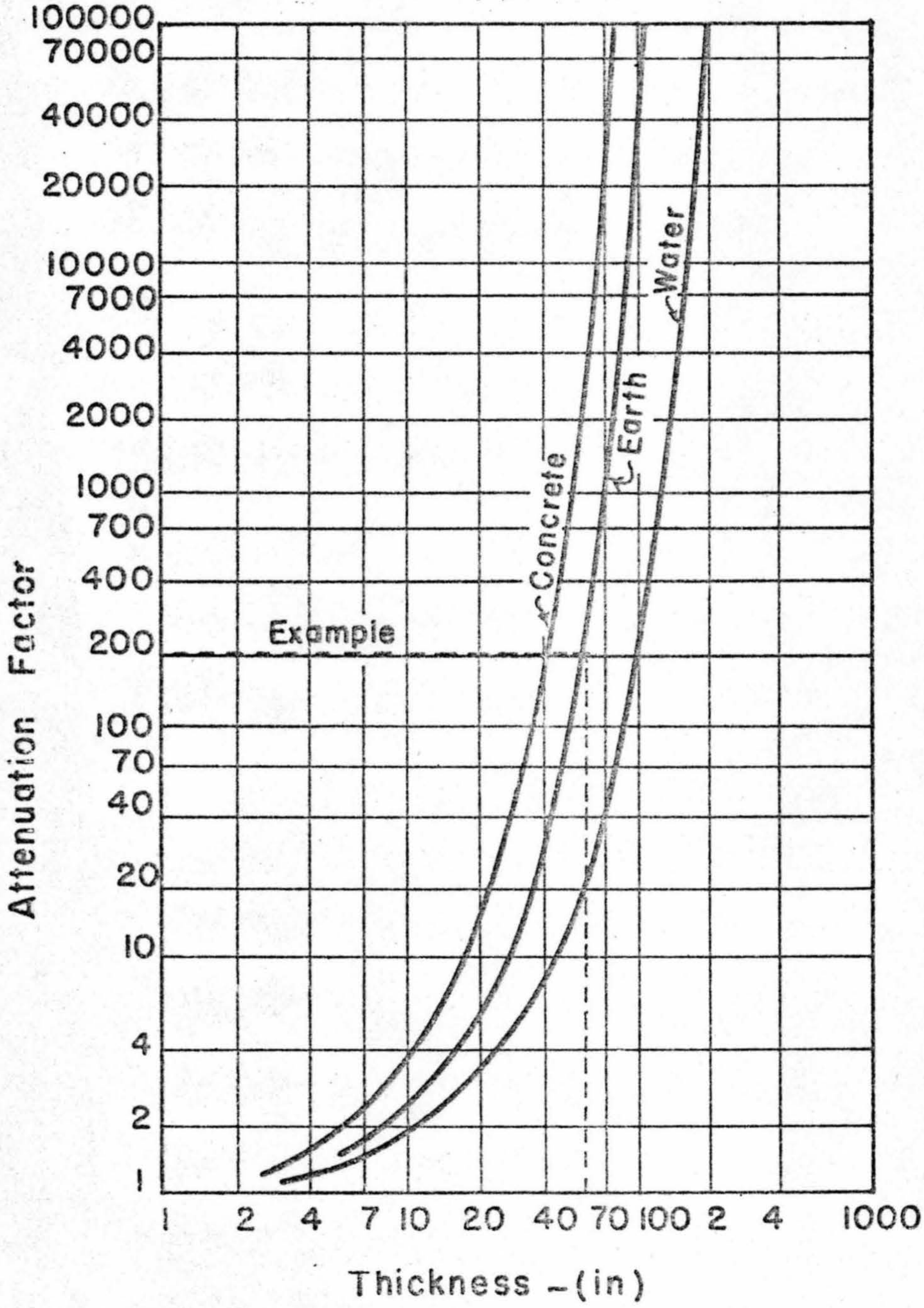


FIG. 6 ATTENUATION OF INITIAL γ RADIATION

initial radiation dosages. Fig. No. 6 gives the required thicknesses in function of the attenuation factor. Example: at 1.5 miles total initial radiation dosage for 20 MT = 2000 KT bomb ≈ 2000 REM which should be reduced to 10 REM, attenuation factor = 200 which requires ≈ 60" of earth. (For more details see "Effects of Nuclear Weapons").

It should be noted that the above mentioned simplified method gives merely an approximate value for the required shielding thickness, since γ rays and neutron radiations are different in nature, but for structural design purposes it yields useable values.

Residual nuclear radiation is defined as a radiation emitted 1 minute after the explosion. This arises from the nuclear radiation of the contaminated earth and bomb particles. With surface explosion, which is, as it has been mentioned earlier, the most critical for structural design, there is not a definite line between initial and residual radiation. Approximate residual radiation for 1 Hr reference dosage rate is given in Fig. 7. For bomb yields other than 1 MT, use can be made of

$$R = R_o \sqrt[3]{C} \qquad \text{and} \qquad d = d_o \sqrt[3]{C}$$

scaling law, where R_o is the 1 hr dose rate for 1 MT at a distance d_o. (For c see previous example). For the required shielding thicknesses curves shown in Fig. 6 might be used, although the shielding properties of materials differ somewhat for initial and residual radiation. Needless to say, air-intakes should be designed in such a fashion that they provide protection against overpressure as well as against radiation. How the latter objective can be achieved by filtering the air is not within the scope of this discussion. It should be repeated again that a 10-20 ft earth cover on the structure provides an adequate and inexpensive thermal

Residual Radiation For 1 MT Surface Blast (W_o)
Assuming Effective Wind

Dose Rate Roentgen / hr.	Radius of G.Z. Circle (miles)	Downwind Distance (miles)	Crosswind Distance (miles)	Displacement of center of G.Z circle (miles)
3000	0.43	22	3.1	0.60
1000	1.4	40	6.8	0.80
300	2.8	70	11.8	1.02
100	4.7	114	16.8	1.24
30	7.5	183	22.8	1.46
10	11.0	317	34.1	1.65

Scaling Low $R = R_o \sqrt[3]{c}$ $d = d_o \sqrt[3]{c}$ $c = \frac{W}{W_o}$

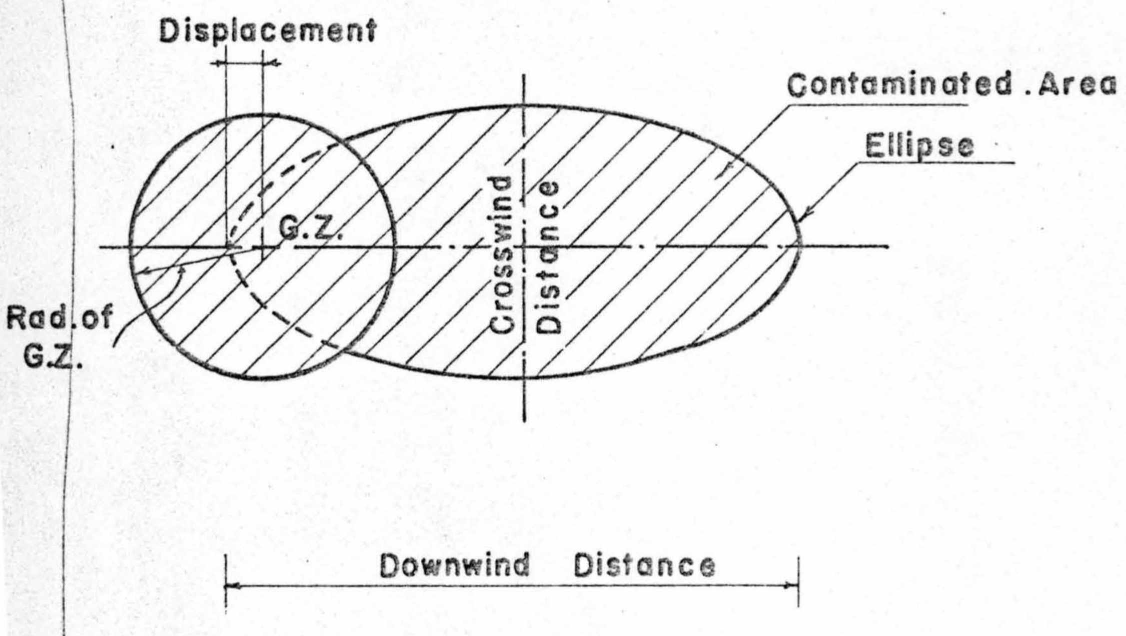


FIG. 7 RESIDUAL RADIATION VALVES & PATTERNS

and nuclear radiation protection for a 20 MT bomb at 1 mile from ground zero.

The explosion produces ground shock waves and reflected shock waves but at overpressures over 100 #/in^2 these waves are of little or no significance.

Before treating the structural design aspects of the atomic shelter design, it should be mentioned what type of material is desirable for such an underground structure. Plastic materials, like reinforced concrete, structural steel can undergo considerably plastic deformation without collapse. The energy absorbed by plastic deformation increases the structural damping effects of structures subject to dynamic loads. Fig. 8 illustrates the stress and strain diagram of an "ideal-plastic" material.

In some cases, a complete protection is required even at ground zero. For example, the design of command control buildings for missile bases require such a protection since they must function even if the enemy hits the target. In such cases, deep tunnels are the most economical solution, especially if they are bored through solid rocks. The external overpressures are distributed by the soil or rock. Since the attenuation by increased depth is very significant, deep tunnels are subject only to the conventional pressures diagram produced by disturbing the geostatic equilibrium by boring the tunnels. Fig. 9 shows pressure diagrams for soils in which plain of failure will be formed. The side pressures exerted on the tunnel wall are due to p_2 surcharge plus active soil-pressure corresponding to plain of rapture. For homogeneous rocks Protogyanokov has derived a simple formula based on experimental measurements (Fig. 10). There are also ground shock waves which should be considered in design. It is, however, a very complex procedure to evaluate the effects of ground

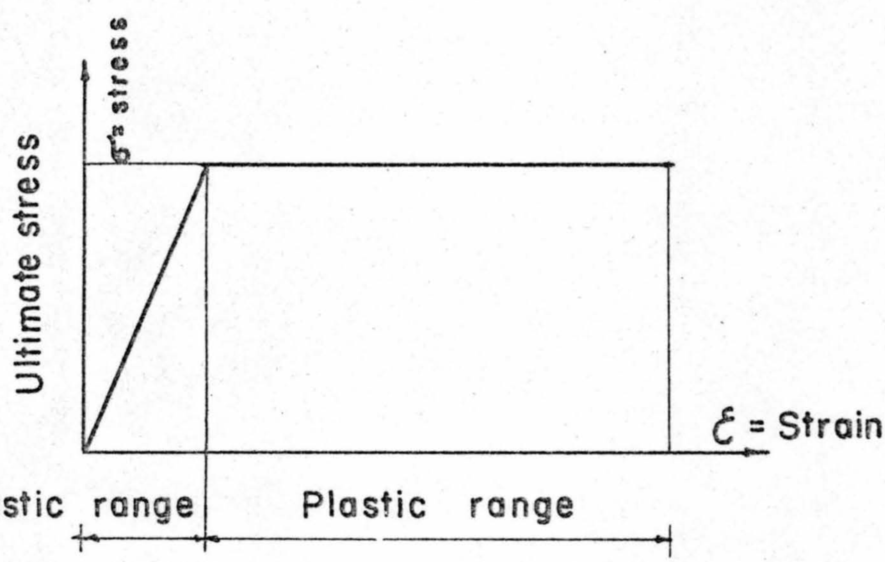


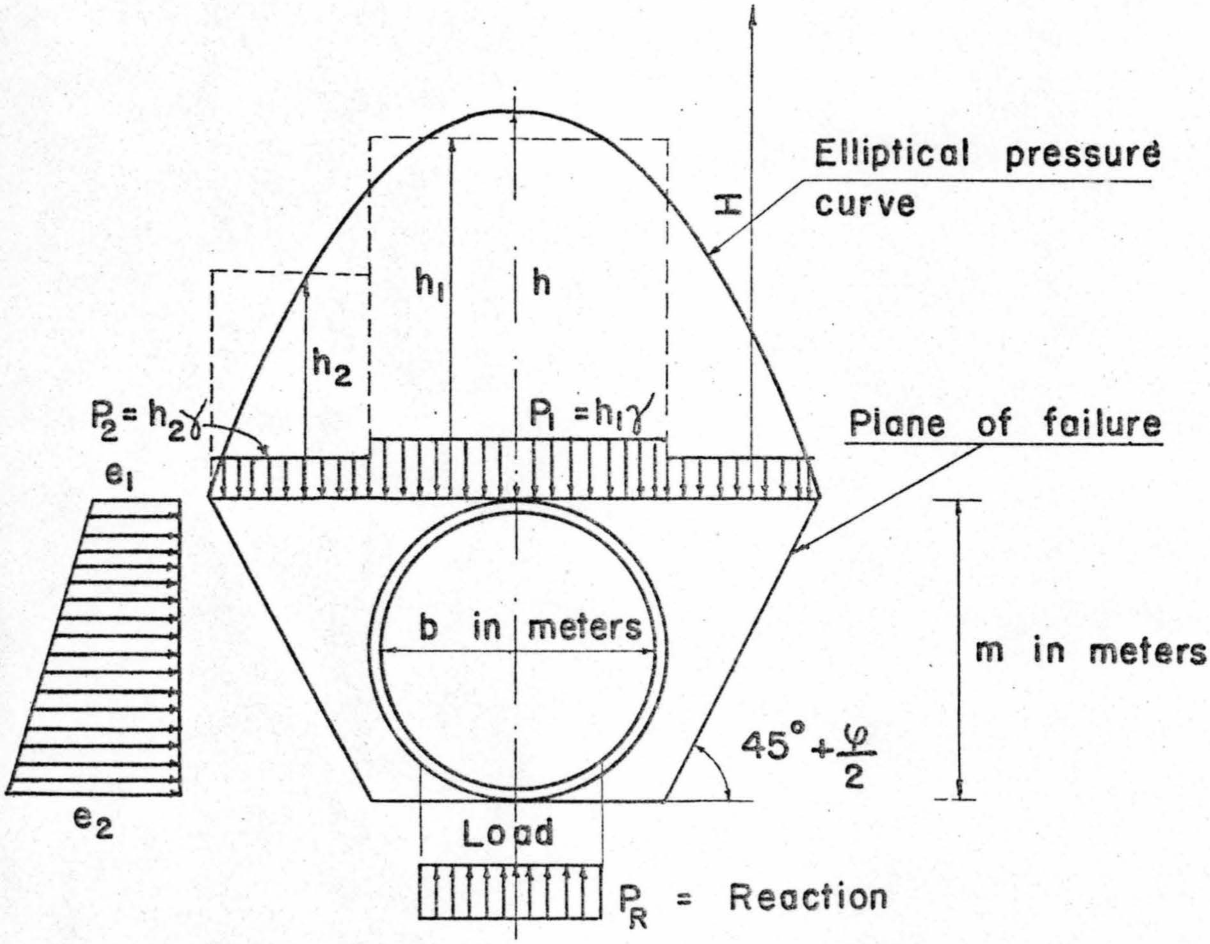
FIG. 8 STRESS-STRAIN DIAGRAM OF IDEAL PLASTIC

$$h = H\left[1 - \frac{tg\,\varphi \times tg^2(45° - \frac{\varphi}{2})}{b - 2m \times tg(45° - \frac{\varphi}{2})}\right]$$

h = Distance from surface

φ = Angle of repose

γ = Specific weight



$$e_1 = P_2\, tg^2(45° - \frac{\varphi}{2}) - 2c.tg(45° - \frac{\varphi}{2})$$

$$e_2 = (P_2 + m\gamma)\, tg^2(45° - \frac{\varphi}{2}) - 2c.tg(45° - \frac{\varphi}{2})$$

c = cohesion

FIG. 9 PRESSURE CURVES ON TUNNELS (IN MATERIALS WITH PLAIN OF RUPTURE)

h = semi-major axis of pressure diagram

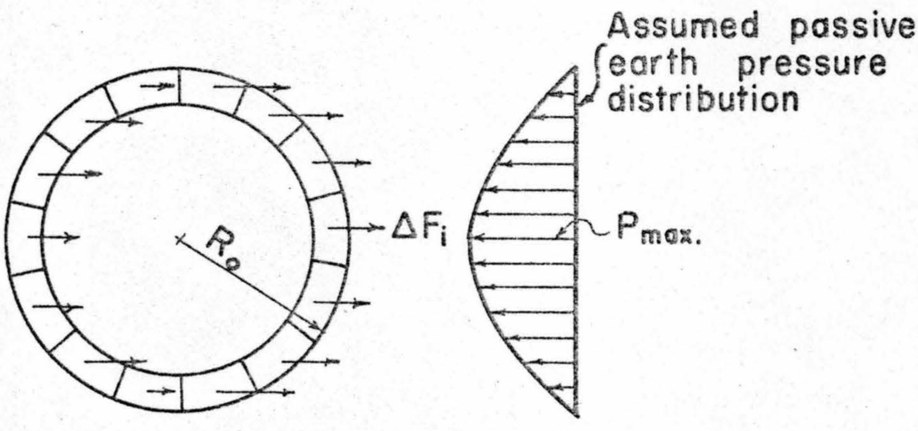
$\cong \dfrac{b}{2f}$

where b = width of tunnel in meter or cm.

$f = \dfrac{\sigma_{ult}\ kg/cm^2}{100}$

σ_{ult} = ultimate compressive strengt of rock.

FIG. 10 PRESSURE DIAGRAM IN ROCKS AFTER PROTOGYANOKOV



$\Delta F_i = c\, \Delta W_i$

$c = 0.1 \rightarrow 0.16$

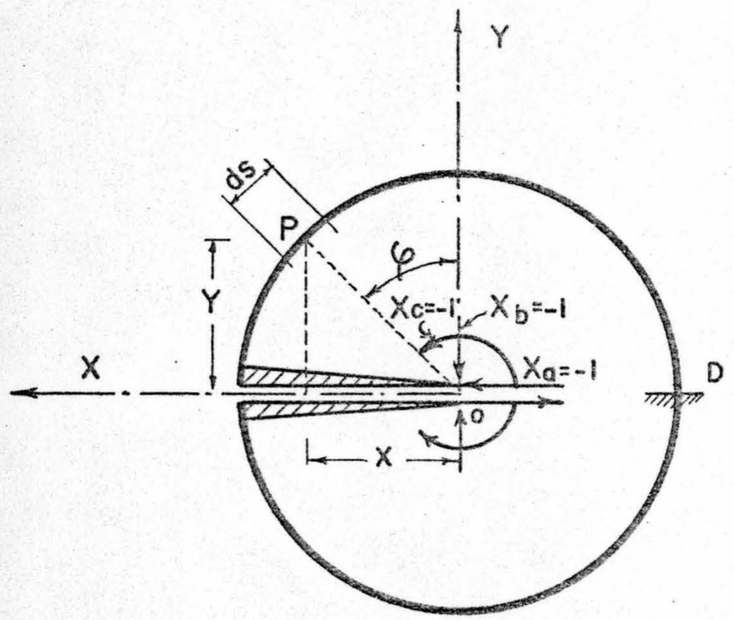
ΔW_i = Weight of one element

$P_{max.} \cong \dfrac{3 \Sigma F_i}{8 \pi R_o}$

FIG. 11 APPROX. PRESSURE DIAGRAMS DUE TO HORIZONTAL SHOCKWAVES shock waves because of the many uncertainties like soil, water content of the soil, effects of depth, etc. involved. The effects of ground shock waves can be approximated by assuming a moderate earthquake (Fig. 11) which may act in any direction in order to produce maximum stresses.

Knowing the external loads acting on the tunnel section, the redundant forces can be expressed by means of virtual work assuming that the structure acts within the elastic zone (Fig. 12). The factor of safety which should be used varies with the load. For earth and water pressures and other conventional loads, safety factors specified in reinforced concrete and structural steel building codes should be used. For atomic blast load which occurs probably once or twice in the lifetime of the structure, a reduced safety factor of 1.3 to 1.5 is warranted.

The majority of underground structures require only a complete protection between 0.7 miles to 1.5 miles from "Ground Zero". Consequently, they use a shallow burial of 5 ft to 20 ft. In such cases, the only surcharge loading will be from pseudostatic overpressure since burial eliminates any effects from directional dynamic pressure. The roof loading becomes simply the overpressure plus weight of the soil and water above the structure. The sidewall loading due to atomic explosion depends on the soil properties; it may be as low as 15% of the overpressure in dry well compacted soils and may reach 100% porous saturated soil. Since we are designing our structures for the most critical condition, the best procedure is to assume saturated earth condition and compute the lateral earth pressure with p_s surcharge adding to it the water pressure also. The floor loading is approximately the reaction of the total external loads. The effects of shock wave compared with the



$$X_a = \frac{\oint M_o Y ds}{\oint Y^2 ds}$$

M_o = Moment of external load on statically determined structure

$$X_b = \frac{\oint M_o x ds}{\oint x^2 ds}$$

$$X_c = \frac{\oint M_o ds}{\oint ds}$$

FIG. 12 REDUNDANTS OF CIRCULAR TUNNEL SECTION other loads is negligible.

Fig. 13 shows a conventional two story, flat slab, reinforced concrete underground shelter subject to 100 $\#/in^2$ overpressure, water and soil pressures. The floor is built monolitically with the sidewalls in spite of the fact that high strength concrete (f_c^1 = 5000 $\#/in^2$) has been used the required dimensions are excessive.

A circular tunnel section represents a more economical section since the external loads are carried mostly by axial stresses. If the wall of the tunnel is relatively rigid, which is always the case in reinforced concrete tunnel design, the pressure diagrams due to earth pressure can be computed according to the equations given in Fig. 14a. Vertical and lateral pressures due to blast overpressure are illustrated in Fig. 14b. The lateral pressure can be computed by conventional methods using P_s overpressure as surcharge load on the ground surface. The distribution of reactive forces depends upon the bedding angle α_o and it can be determined by following the formula given in Fig. 14c, in which Q represents the resultant of all the external forces acting on the section. The redundant forces can be determined by using equations shown in Fig. 12.

Another economical solution for tunnels not over 15 ft of diameter is the use of flexible conduit type structures. The flexible conduit type structure would fail by excessive deflection rather than by rupture of the pipe wall, (Fig. 15). In order to take care of the high overpressure they should be made of double walled corrugated metal at least 3/8" thick. Between the external and internal skin a weak grout, consisting of 1 part cement and 8 part of builders sand shall be placed,

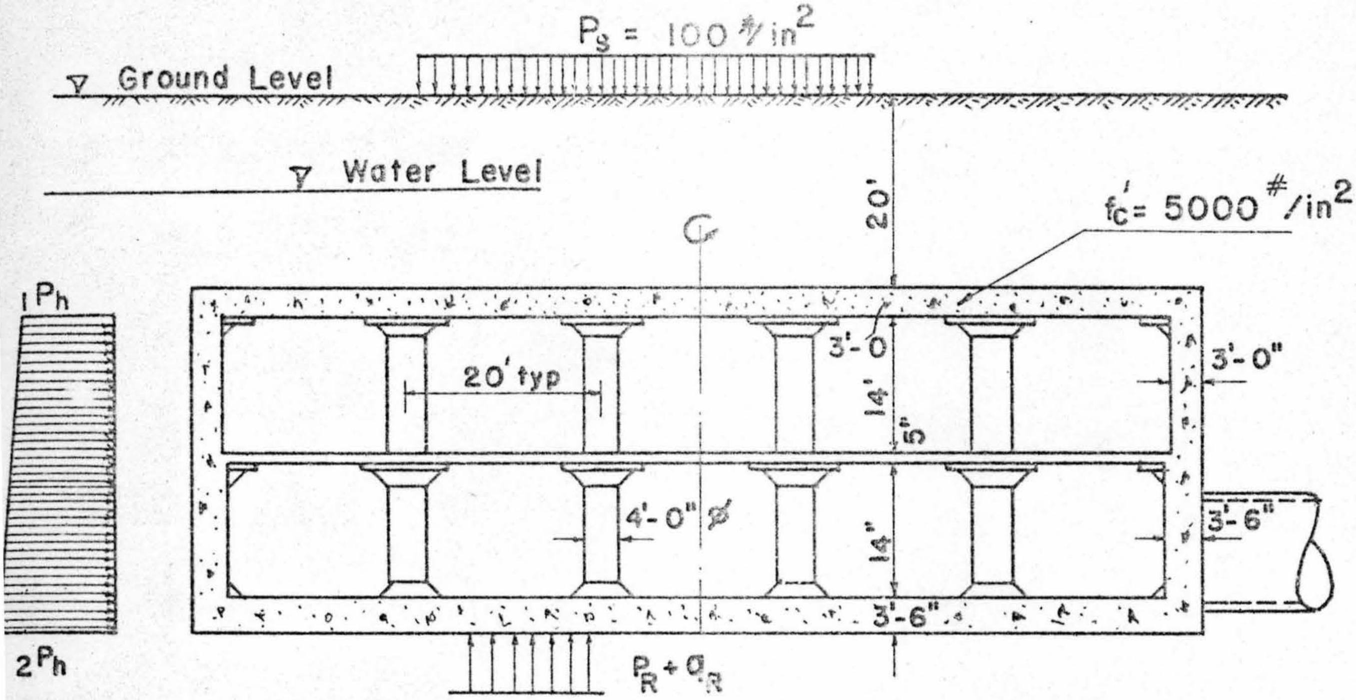
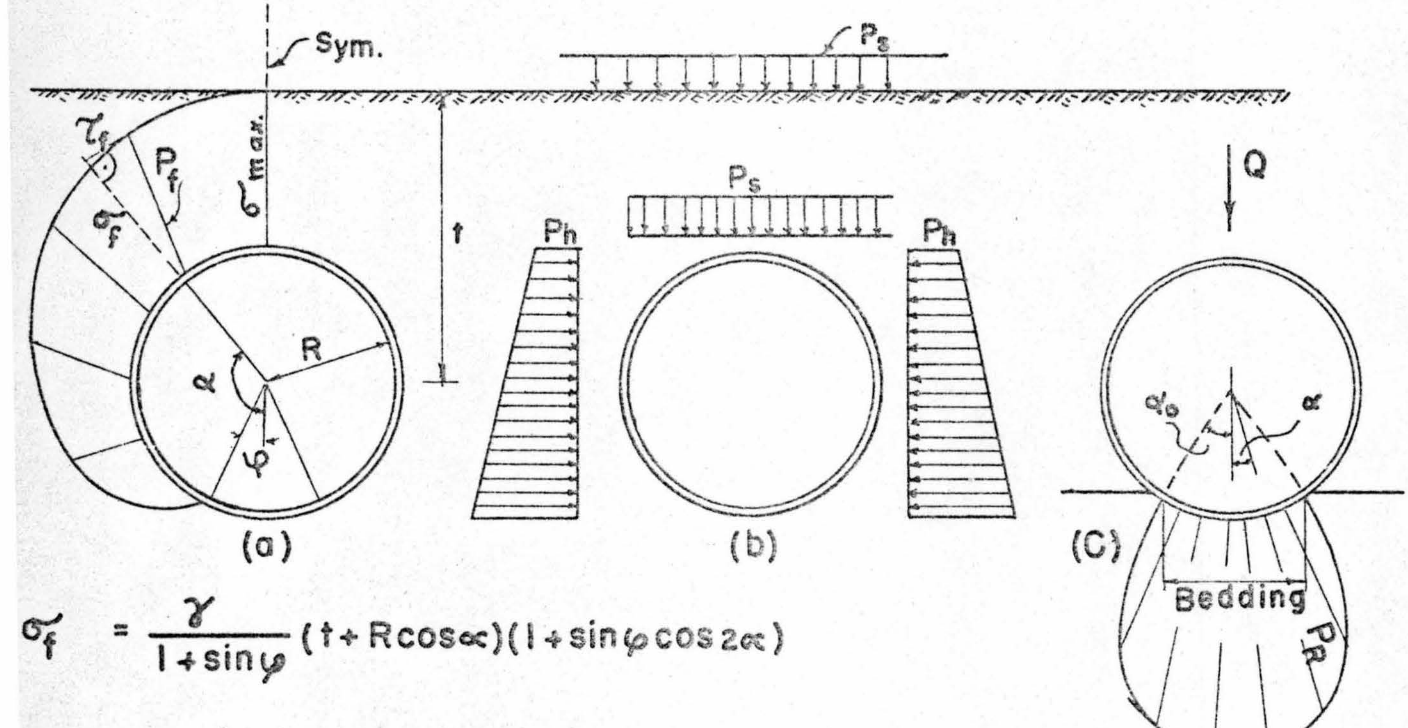


FIG. 13 CONVENTIONAL STRUCTURE



$$\sigma_f = \frac{\gamma}{1+\sin\varphi}(t+R\cos\alpha)(1+\sin\varphi\cos 2\alpha)$$

$$\tau_f = \frac{\gamma}{1+\sin\varphi}(t+R\cos\alpha)\sin\varphi\sin 2\alpha$$

$$P_R = \frac{3Q(\cos\alpha-\cos\alpha_o)\cos\alpha}{R(3\sin\alpha_o+\sin^3\alpha_o-3\alpha_o\cos\alpha_o)}$$

$\sigma_{max.} = (t-R)\gamma$ γ = wt. of earth Q = Resultant of ext. loads

FIG. 14 PRESSURE ON RIGID CIRCULAR SECTION

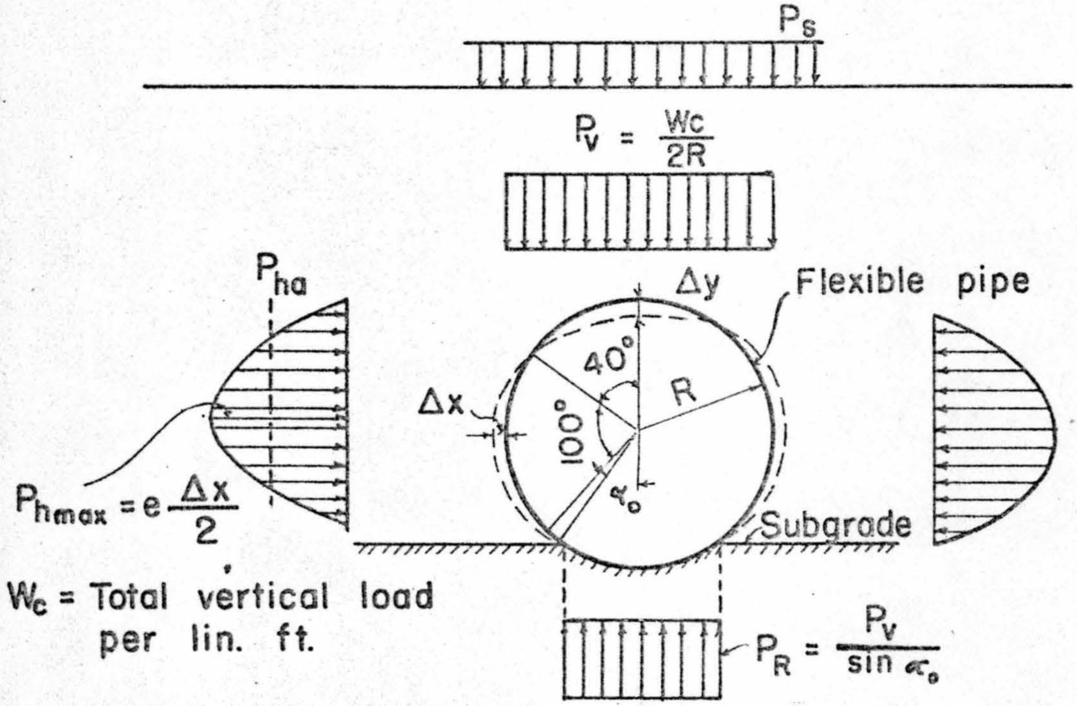


FIG. 15 PRESSURE DISTRIBUTION ON FLEXIBLE CONDUIT TYPE SECTION

which would not interfere with the deflection of flexible pipe.

It can be safely assumed that each skin carries approximately 50 per cent of the total load. The active and passive earth pressure diagrams are shown in Fig. 15. In the formula of the horizontal earth pressure "e" represents the modulus of passive resistance of the side filling materials. It is defined as a unit pressure developed as the side of structure moves outward a unit distance against the sidefill. This modulus varies widely with the soil characteristics, grade of compaction and moisture content. The low values of "e" are between 4 to 8 lb. per in^2 per in the highest values are in the vicinity of 50 psi. per in. For well graded compact material, which should be used around the tunnel, "e" can be assumed 35-40 psi. per in., for preliminary computation purposes. For final design actual values should be determined for different moisture contents.

In order to determine the horizontal passive earth pressure the horizontal deflection should be also computed using the following formula developed by Spangler.

$$\Delta x = D_1 \frac{K\ W_c\ R^3}{EI + 0.061\ e\ R^4}$$

in which

Δx = horizontal deflection of the pipe in inches

D_1 = deflection lag factor due to long-time yielding of the ambient soil (1.25-1.50 for dead load and 1.0 for blast load)

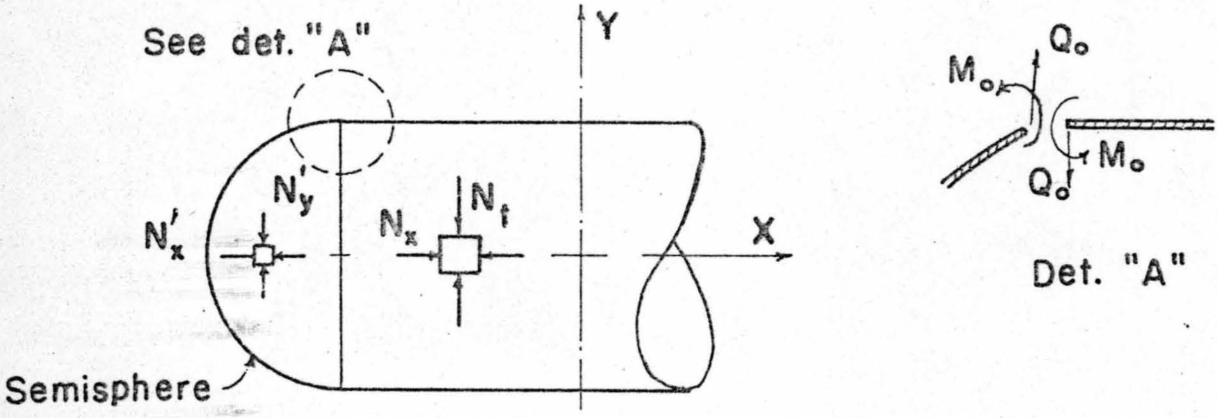
K = a bedding constant, its value depending on the bedding angle. (at $\alpha = 45°$ K = 0.096 it varies from 0.110 at $\alpha = 0$ to 0.083 at $\alpha = 90°$.

I = modulus of inertia per unit length of cross section of the conduit wall.

W_c = total vertical load per unit length of conduit.

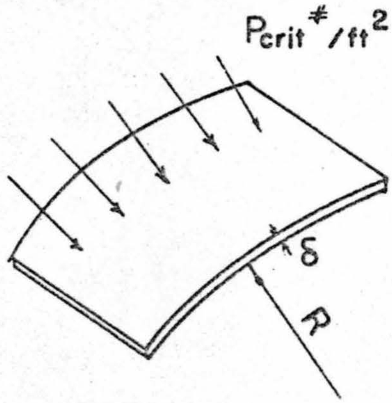
Maximum allowable total horizontal deflection is approximately 10 per cent of the diameter, since blast loads are included. For conventional loads a maximum of five per cent is recommended. Analysing the structure as a ring girder using the methods of elasticity, one shall find that the stresses due to the moments are well over the elastic limit of the material. Consequently, the conduit type section acts as a three-dimensional thin shell structure developing a membrane stress condition. Its characteristic feature is that the external loads are carried by direct stresses, since the structure is too thin to develop considerable moments. The typical difference between shell and beam action will be explained in more detailed form later on.

Fig. 16 illustrates the meridional and the ring forces in the cylindrical as well as in the spherical part of the thin shell subject to uniform external pressure. Negative sign indicates compression according to the customary sign convention. At the boundary between cylindrical and spherical shell, where the radius of curvature changes abruptly, moment disturbances are expected but they attenuate very rapidly. The elastic stability against local buckling should be checked carefully. In most of the cases the critical loading is the local buckling loading which is given in Fig. 17. The total failure of the structure occurs due to excessive deflection of the crown. The top becomes approximately flat and small additional load causes the curvature of the top to reverse direction becoming concave upward, the sides pull inward



$$N_x = -\frac{P \times R}{2} \qquad N_t = -P \times R \qquad N'_y = N'_x = -\frac{P \times R}{2}$$

FIG. 16 MEMBRANE STRESSES ON FLEXIBLE CONDUIT TYPE SECTION



$$P_{crit} = \frac{E}{4(1-\mu^2)}\left(\frac{\delta}{R}\right)^3 \quad = \text{BucKling load}$$

E = Modulus of Elasticity μ = Poisson's ratio

FIG. 17 BUCKLING OF CYLINDRICAL SHELL

and the structure rapidly collapses.

It is evident that the total carrying capacity of the structure depends largely upon the sidefill material. Thus, fill must be considered as integral part of the structure. Fig. 18 shows the recommended excavation and backfill methods for flexible conduit type shelters. Backfill "A" should consist of well graded and well compacted gravel, (max. grain size 1 1/2") placed in 1.5' thick layers. Fill "B" which envelopes the structure should consist of a finer (max. grain size 1/2") hand tamped material, whereas fill "C" on the top of the structure consists of essentially the same material as fill "A", but it should be compacted by means of watering.

Flexible conduit type structures represent a large saving in construction cost compared with conventional structures, but their application is limited to smaller longitudinal type structures. For larger units the double curved rotational type thin shells offer remarkable advantages. As it has been mentioned before the main difference between beam and shell action consist of the fact that beams carry external loads by means of internal shear and flexural stresses whereas a shell (Fig. 19) transmits the external loads by direct stresses (tension, compression, and shear) which are generally called membrane stresses. The moments are negligible except at the openings and edge beams where the membrane condition is disturbed. Fig. 20 illustrates an elliptical shell of revolution indicating the values of the meridional and hoop forces at the crown and at the equator assuming uniformly distributed rotational symmetrical loadings. Using the same sign convention as previously: negative sign indicates compression and positive sign tension. The prerequisite for

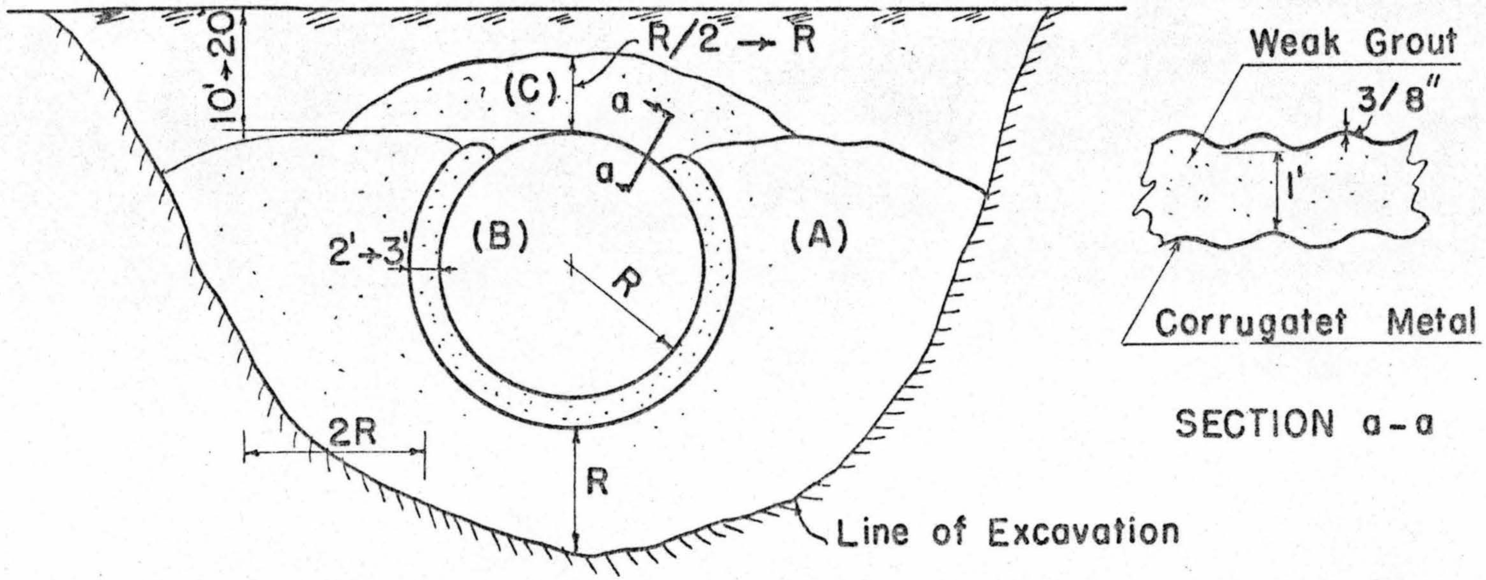
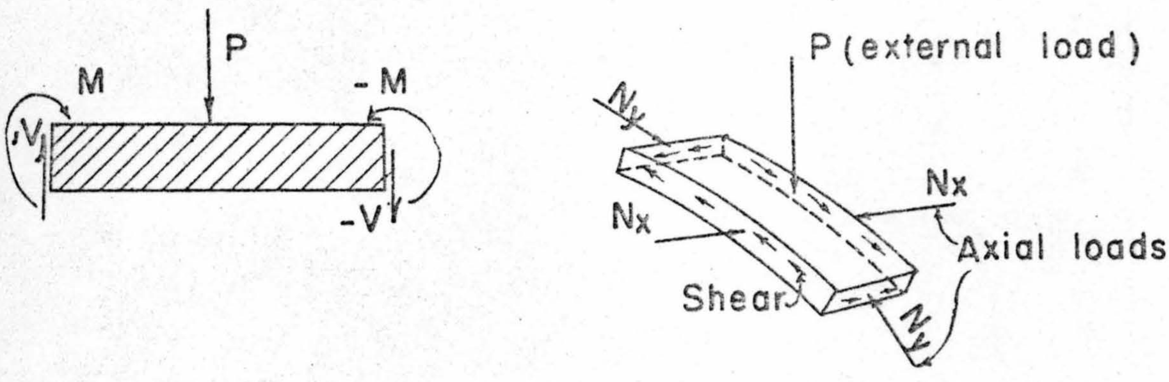
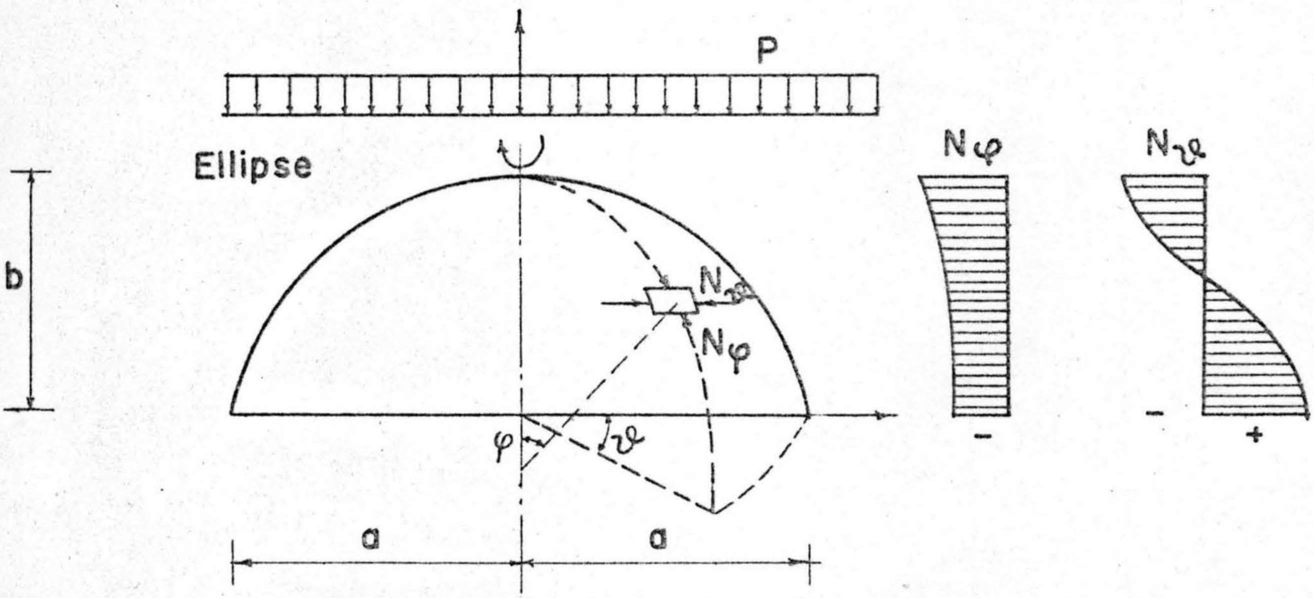


FIG. 18 EXCAVATION & BACKFILLS FOR FLEXIBLE CONDUIT TYPE SHELTER



(a) Beam (b) THIN SHELL

FIG. 19 BEAM ACTION VS. SHELL ACTION



At crown $N_\varphi = N_\vartheta = -\dfrac{P\ a^2}{2b}$

At equator $N_\varphi = -\dfrac{P\ a}{2}$ $N_\vartheta = +\dfrac{P\ a^3}{2b^2}$

FIG. 20 INTERNAL FORCES IN ELLIPSOID OF REVOLUTION this very desirable membrane stress condition is that the thickness of the shell is small in comparison to the major and minor axes of the dome.

The recommended general configuration of the large underground atomic shelter is an egg-shaped dome which protects an entirely independent, multistory, internal frame structure (Fig. 21). The external shell can deform freely under the rapidly applied blast load increasing the ultimate load carrying capacity of the structure by developing passive earth pressures. The conventional internal frame is supported on compressible foundation material. The connection between the external shell and internal rigid-frame is of flexible type allowing free deformation for the shell. The wall thickness of such a shell is between 14" and 16" for spans up to 130' using pneumatically placed concrete like "gunite".

Design experience indicates that the elastic stability in form of local buckling is critical for thin shells under heavy blast loads, Fig. 22. The external load distribution can be assumed as uniformly distributed for loads acting on the top and bottom of the shell. The magnitude and distribution of sideloads are uncertain. For a conservative assumption the sideloads may follow a trapezoidal distribution based on the active soil pressure with surcharge. In this respect further research is required. It is expected that the lateral pressure distribution will have a close similarity to that of the flexible conduit type. It can be safely approximated by a uniformly distributed sidewall loads are 50 to 70 per cent of the vertical load.

In order to reduce the danger of buckling in some cases stiffened

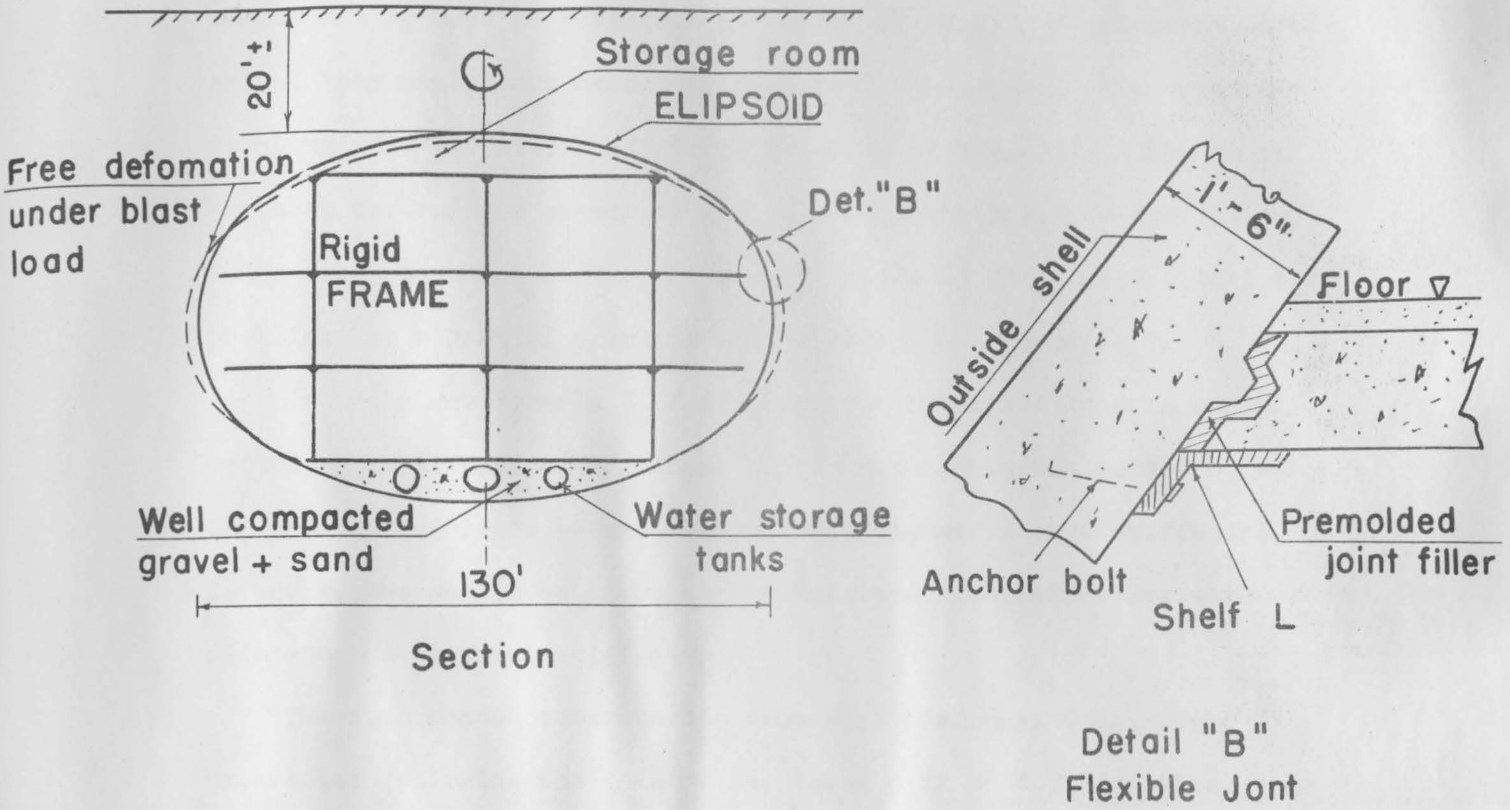
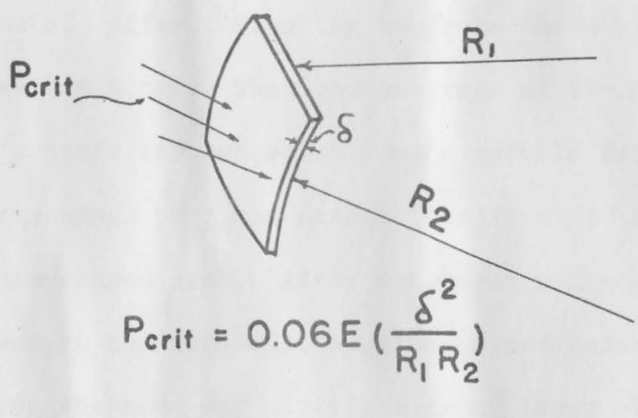


FIG. 21 LARGE THIN SHELL SHELTER



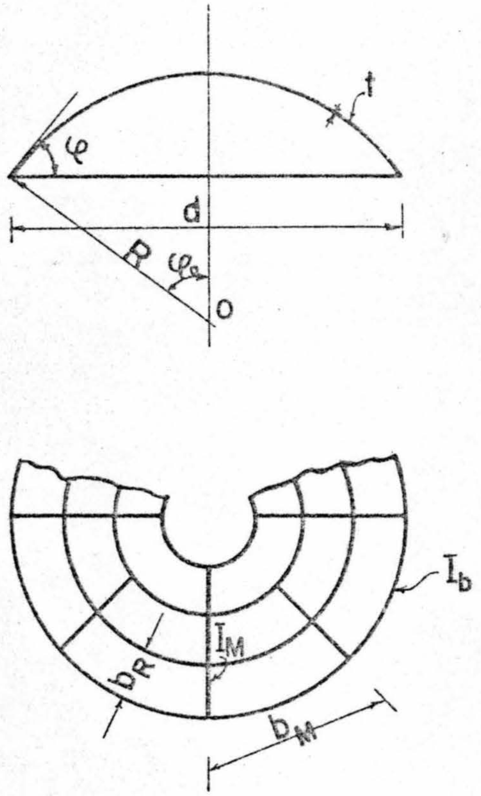
$$P_{crit} = 0.06E\left(\frac{\delta^2}{R_1 R_2}\right)$$

FIG. 22 BUCKLING OF DOUBLE CURVED SHELL

shells can be used to advantage in design of reinforced concrete structures; they are almost mandatory for steel structures. The critical buckling load for stiffened shells is given in Fig. 23. Although it has been derived for spherical shells but can be utilized for ellipsoids also, assuming that the top part, which is the most critical for buckling, is a part of a sphere with a radius of curvature R.

In the given formula I_M represents the moment of inertia of the meridional stiffener with respect to its own centroidal axis. b_M = the greatest distance between the neighboring meridional stiffeners. I_R and b_R are analogous to the above mentioned properties but with reference to the ring stiffeners.

The recommended construction procedure commences with proper excavation following the form of the lower part of the elliptical dome (Fig. 24). The earth or subgrade must be firm enough to sustain pressure without considerable yielding. Placing a wiremesh the first layer (3") of concrete can be pneumatically placed against the ground leaving the inside surface rough. Placing the reinforcing the other layers can be placed in horizontal rings proceeding from the lower crown towards the major axis. After curing the concrete the construction of the internal frame can start. When the concrete of the inside frame is strong enough to support its own weight and a movable framework the construction of the upper part can start. The first 3" thick layer of concrete form a pie-shaped arch. After concrete is strong enough to support its own weight the form will be lowered and rotated into position No. 11. Proceeding the same way until the first layer of dome completed, which is going to provide a perfect support for the second and third



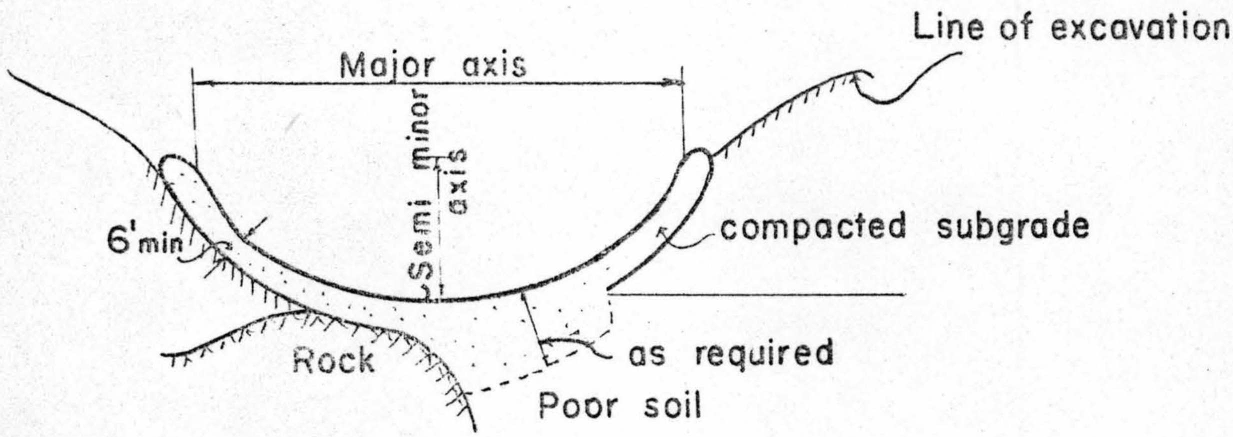
$$P_{crit} = 1.09\,\frac{E\sqrt{t_I^3 \times t_F}}{R^2} \qquad \text{valid for } 20^\circ \leqq \varphi_0 \leqq 60^\circ$$

$$t_I^3 = \frac{t^3}{0.89}\,\xi\left(0.8\,\frac{d}{2b_M + b_R} - 0.2\right)\left(1 - 0.4\,\frac{\varphi_0 - 20^\circ}{20^\circ}\right)$$

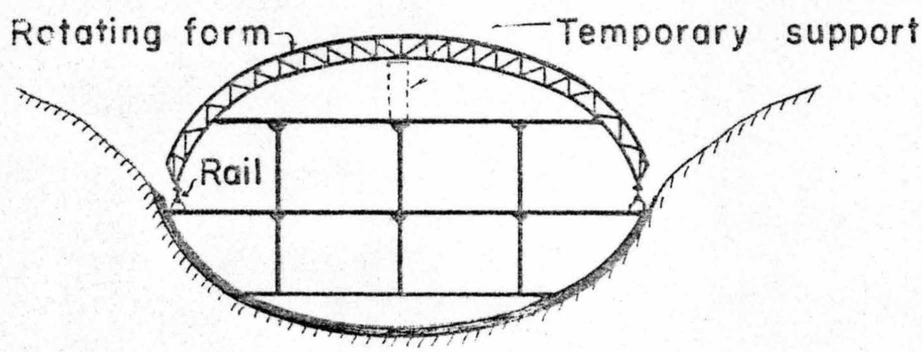
$$t_F = \frac{t}{0.89} + 0.5\,\zeta$$

$$\xi = \frac{I_M}{b_M} + \frac{I_R}{b_R} \qquad \zeta = \frac{F_M}{b_M} + \frac{F_R}{b_R}$$

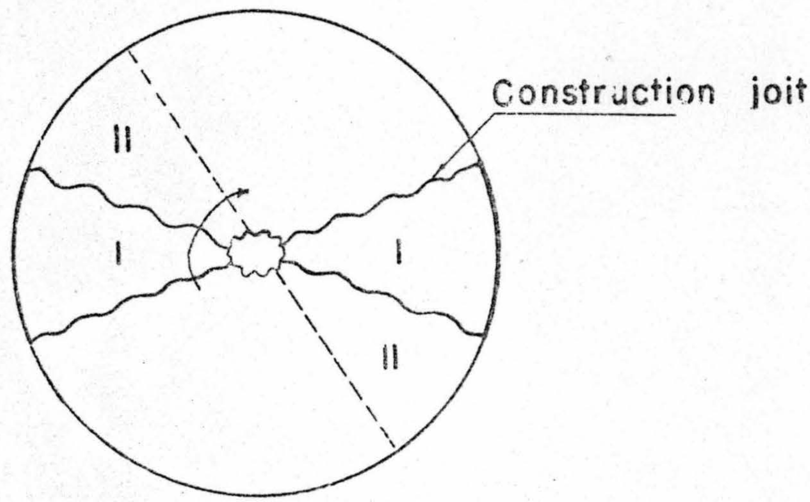
FIG. 23 BUCKLING OF STIFFENED SPHERES



ELEVATION



SECTION



PLAN

FIG. 24 CONSTRUCTION SEQUENCES

layers of concrete.

It is hoped that the foregoing introductory treatment of a broad subject has furnished some basic understanding covering the design of underground shelters for nuclear blast loads.

Finally, the writer would like to emphasize the fact that further research is required in this field covering especially the behavior of structural materials (including earth) under suddenly applied nuclear blast loads.

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