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ROCK MECHANICS APPLIED TO THE DESIGN OF UNDERGROUND INSTALLATIONS TO RESIST GROUND SHOCK FROM NUCLEAR BLASTS

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MINING RESEARCH CENTRE

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ROCK MECHANICS APPLIED TO THE DESIGN OF UNDERGROUND INSTALLATIONS TO RESIST GROUND SHOCK FROM NUCLEAR BLASTS

by

D.F. Coates*

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SUMMARY

Underground installations designed to resist the effects of nuclear explosions are generally of a permanent nature and hence cannot be treated exactly like mining openings. For example, scaling of walls and backs may not be possible after the installation is completed; rehabilitation and replacement of sets in deteriorated sections generally is not possible; support such as rock bolting must resist corrosion. Also, the shape of openings cannot normally be modified after experience has indicated the nature of ground reaction.

The phenomenology of nuclear explosions is now fairly well known. Effects such as the various types of radiation, fallout, air blast and ground shock can be predicted for engineering purposes. For underground installations, ground shock, or the dynamic stress wave, produced by a nearby explosion provides one of the main design conditions for the main openings and their associated service entrances and exits as well as for the structures and equipment contained within the openings. Beyond a certain range these effects can be provided for through dynamic design methods. However, within a certain range it almost becomes impossible to provide protection against the high-intensity ground stress and motion that is created by the explosion.

Whereas designs for such installations can be made, many assumptions are required that should be examined by research work. The majority of such research should be on the behaviour of rocks and structural systems under dynamic loading, which can be done without necessarily using nuclear explosives.

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ROCK MECHANICS APPLIED TO THE DESIGN OF UNDERGROUND INSTALLATIONS TO RESIST GROUND SHOCK FROM NUCLEAR BLASTS

D. F. COATES

1. INTRODUCTION

ANALYSES of some of the problems facing the designer of underground installations that are required to resist the effects of nuclear explosions are presented. The problems that are considered are restricted to those involving rock mechanics. In addition, consideration is only given to those installations requiring relatively large chambers.

Underground installations of this nature are generally permanent. Hence they cannot be treated exactly like mining openings. For example, scaling of walls and backs may not be possible after the installation is completed; rehabilitation and replacement of sets in deteriorated sections generally is not possible; support such as rock bolting must resist corrosion. Also, the shape of openings cannot normally be modified after experience has indicated the nature of ground reaction.

Many of the analyses involve the use of elastic theory, which provides a point for criticism. However, aside from the fact that there is little other theory that is as serviceable, use of elastic theory has some justification. For dynamic loads many rocks produce straight line stress-strain curves, at least on the loading cycle, which generally satisfies the principal requirement of elasticity. Also, other ground reactions, e.g. visco-elastic, plasto-elastic, elasto-plastic, etc., can be considered as modifications of the answer obtained from the elastic solution. The solutions for these other materials must still include the same equilibrium equations and boundary conditions. The compatibility equations will be different. Hence the elastic solution can always be considered as a first approximation and, if necessary, extrapolation can be by judgment.

2. PHENOMENOLOGY OF NUCLEAR EXPLOSIONS

Underground installations may fail as a result of nearby nuclear explosions for many reasons. The ground shock that is produced may cause the rock around the underground openings to rupture. Or the ground motion associated with the shock wave may cause failure, similar to the damage caused by earthquake motion, of structures, instruments or equipment located within the opening. Also, the connections with outside services may be cut off. For example, air, water and communication intakes together with exhaust, effluent and communication outlets may be damaged by rock failure in the passages containing these services, by failure of pipes and cables as a result of ground motion or by air blast causing damage.

It can be shown that ground shock effects are most likely to damage an underground installation. It follows then that a ground surface burst is more likely to cause damage than an air burst. The installations required to resist the effects of such explosions should therefore be designed for surface bursts of a probable magnitude.

Actually in view of the uncertainty of the magnitude of the explosion and its proximity, an alternate approach is to exploit the natural advantages of a site and, within economic limitations, do everything to make the installation more resistant to all the potential, damaging effects. It may be noted here that although the thermal and radiation effects would be very severe close to such an explosion the resultant design problems for an underground installation are comparatively easily resolved.

The high pressures exerted on the ground near the center of the explosion are sufficient to cause ground failure in the immediate vicinity. A crater is formed with dimensions that vary with the strength of the ground. Figure 1 shows the average crater depth, diameter, lip and rupture zone that would be expected from a surface burst of 1 Mton on hard rock.[1]

Empirical information has been analysed to establish correlations between crater size and magnitude of explosion.[1] It has been assumed that $R_c = K.W^{(1/3)}$, where R_c is the crater radius, K is a constant and W is the yield of the explosion. However, work has shown that taking into account weight and strength the exponent should be 1/3.4.[2] Now more recent work indicates that the exponent should vary with the material.[3] This should be noted by those engaged in blasting research. For a hard rock the diameter of the crater in Fig. 1 may be only 300 ft.[3]

It should be possible to obtain a more accurate estimate of the size of the crater in a particular material by calculating the pressures that result from the explosion, and then by applying bearing capacity theory predict to what distance these pressures will exceed the strength of the surface and underlying ground.

Owing to the high temperatures and kinetic energy in a nuclear explosion it is possible to determine the pressures close-in to the explosion using hydrodynamic theory.[4] Based on this theory some of the crater pressures for a 1 Mton explosion are shown on Fig. 1. At the bottom of the crater the hydrodynamic state changes and the material is then in a solid, probably plastic state.

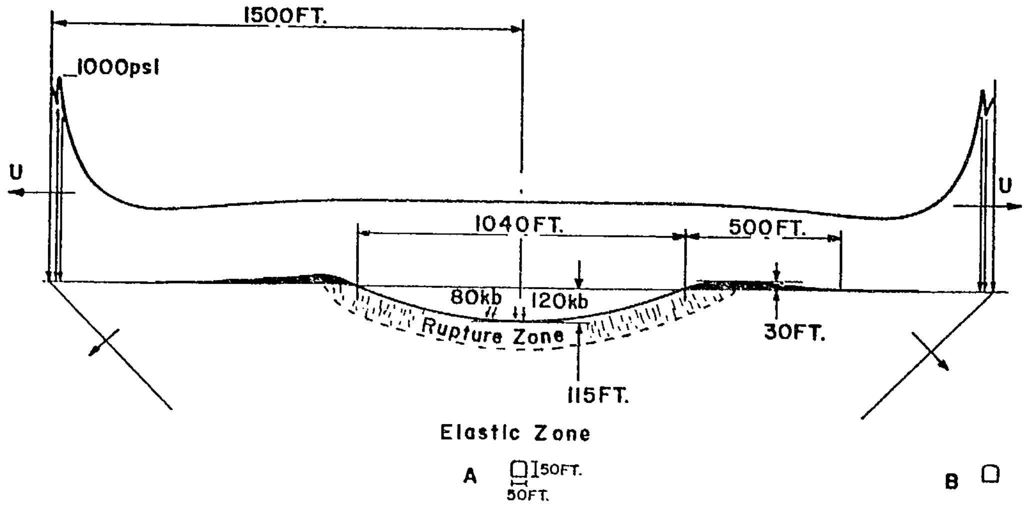

FIG. 1. Crater and air blast from a 1 Mton surface burst.

Pressure will be transmitted through this plastic zone, being attenuated by geometrical dispersion and by plastic action, until it is no longer high enough to cause failure. The pressure then becomes a seismic stress or strain pulse propagating outwards in the more or less elastic ground. As a first approximation the variation of these pressures with distance away from the center of a surface explosion of 1 Mton is shown in Fig. 2.(5)

At some distance away from the explosion the attenuation of the stress pulse in the ground originating from the impulse delivered directly from the explosion products is great enough so that the major ground stress effects arise from the shock pressures that exist in the air and act on the surface of the ground. The air blast existing at one instant is shown in Fig. 1. The variation of the peak air blast pressures with distance is shown in Fig. 2.

The calculation of the magnitude of the stress pulse transmitted into the ground directly from the explosion requires information on the nature of attenuation beyond the crater. The development of a plastic zone outside the rupture zone around a crater has been observed in gravels and tuffs. However, it is questionable whether such a zone would be developed in a harder rock. It is probable that the crater would only be surrounded by a fractured, unscoured zone. Furthermore, in a strong, brittle rock the idealized rupture zone around the crater may either not exist or be very thin. In this case the attenuating effects of this zone could be ignored to obtain an upper limit for the stress pulse that would be transmitted to the ground around an underground opening.

If the position of the installation were 600 ft deep and directly under the explosion as shown by point A in Fig. 1, the attenuation of the peak stress would be proportional to the inverse cube of the distance in the hydrodynamic state and to the inverse square of the distance in the elastic state as a result simply of spherical attenuation. In this case, as seen from Fig. 2, if the dynamic strength of the rock is 12 times the static strength of 30,000

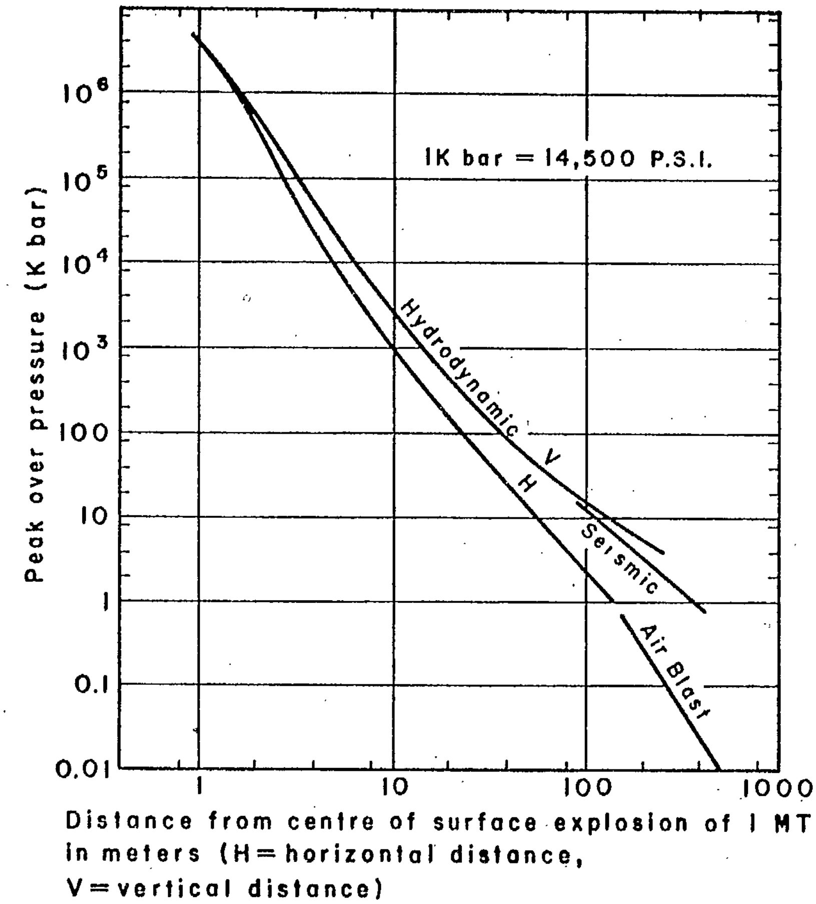

FIG. 2. Variations of peak overpressure in and around the crater.

psi[6] the peak stress at A would be about 3.4 kbar or about 50,000 psi. If the material possesses any viscous properties the reduction of sharp peak stresses can be significant.[7] At the same time it is possible that spherical attenuation does not apply for this case owing to the rapidly expanding area under pressure on the surface;[8] although this consideration would lead one to use the pressure in the central region (see Fig. 1) rather than the peak at the front. However, some experimental work has shown very high attenuation rates close-in.[9]

It can be noted here that at any given depth the peak dynamic stress might decrease with a decrease in strength of surface material. This could be expected simply from the crater being larger and, aside from any other attenuating mechanisms, the inverse cubic attenuation continuing for a greater depth.

3. STRESS DISTRIBUTION AROUND UNDERGROUND OPENINGS

For illustrative purposes an opening 50 ft wide, 50 ft high and 400 ft long will be used. The rock will be assumed to be a uniform granite with a P-wave velocity of 15,000 ft/sec, a static compression strength of 30,000 psi, a modulus of deformation of 8×10^6 psi and a Poisson's Number of 4. The effect of a 1 Mton surface explosion will be examined.

In designing an installation the theoretical stress distribution in the rock around the underground opening arising from static loading should be determined first. From these calculations, assuming elastic deformation, an indication is obtained of the maximum probable compressive stress and of the location and depth of ground likely to be subjected to tensile stresses. The main difficulty in performing these calculations is to know the magnitude of the horizontal stress.

The horizontal stress may be due simply to confinement of the rock in the horizontal direction resisting the tendency to expand owing to vertical gravitational stresses. In this case, the horizontal stress in brittle rock would be related to the vertical stress through Poisson's ratio.[10] In less competent

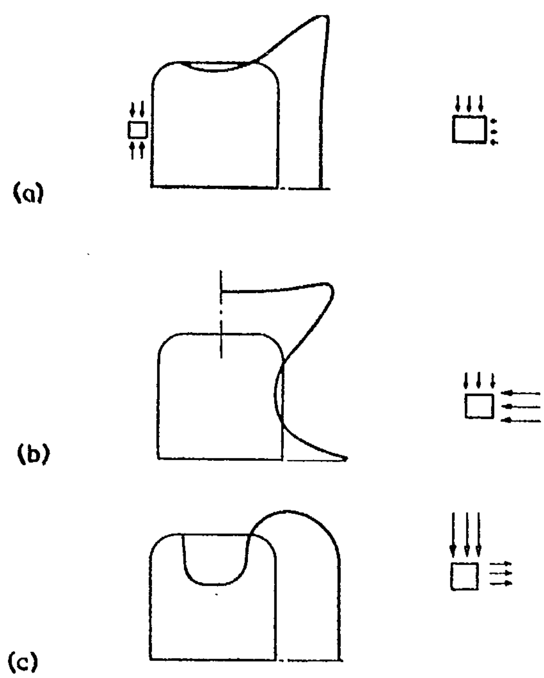

FIG. 3. Static and dynamic stress distributions.

ground the horizontal and vertical stresses may be related through plastic parameters.[10] Alternatively, the major principal stress underground might arise from orogenic action.[11–13] In this case, the horizontal stress is theoretically indeterminant and should be measured in the field.[12,14]

Two extreme cases are shown in Fig. 3. In Fig. 3a the variation of the tangential stress around the opening due simply to gravitational loading is shown. In this case there is a zone of tension in the roof. In Fig. 3b, with an orogenic horizontal stress three times the vertical stress, a tension zone is produced in the walls. In both cases the maximum compressive stresses are not large compared to the strength of hard rocks.

The shock or stress pulse that is created in the ground results from the conversion of nuclear energy into mechanical energy. Of the total energy released in a nuclear explosion only a fraction is converted into kinetic energy—initially of the contained materials and ultimately of the adjacent substances. The balance of the energy is converted into radiation.

By considering the conservation of momentum that must exist for a surface burst between the air and the ground, i.e. the atmosphere must provide the reaction for the push which the explosion exerts on the ground, it can be shown that only a small part of the kinetic or mechanical energy of the explosion can be transmitted into the ground. The calculation below indicates that about 1% of the total energy in the explosion might be transferred into the ground. Some experimental work tends to confirm the answer;(6) however, other work does not.(15)

Based on the conservation of mass, energy and momentum of air and ground during the burst, i.e. Rankine-Hugoniot equation of state(16,17) and assuming equal pressure all around the hot sphere:

$$\frac{\dot{W}_a}{\dot{W}_s} = \left[\frac{\rho_o{}^s}{\rho_o{}^a}\left(\frac{n-1}{n}\right)_a\left(\frac{n}{n-1}\right)_s\right]^{0.5}$$

where $\dot{W}_a$ = rate of shock energy put into air,

$\dot{W}_s$ = rate of shock energy put into soil,

ρ_o = original density,

n = compression under shock pressure.

To determine the average compression to use during the period of partition of the explosion energy, a time average is determined up to the point where failure of the rock ceases. If the dynamic strength of the rock as used in Section 2 is about 24 kbar, the average pressure will be about 100 kbar. The compression of the rock under this pressure can be calculated assuming the modulus of deformation, as determined statically in the laboratory, still applies under these conditions.

$$n_s = \frac{1}{1-\mathrm{P}_{SO}/E} = \frac{1}{1-100\times 14{,}500/8\times 10^6} = 1{\cdot}22$$

where P_{SO} = crater overpressure.

This figure is actually very close to test data obtained for granite under similar pressure.(18,19)

The compression of the air at these pressures is less significant in the calculations and will be assumed to be 50. Hence

$$\frac{\dot{W}_a}{\dot{W}_s} = \left[\frac{167}{0.081}\left(\frac{50-1}{50}\right)\left(\frac{1.221}{1.221-1}\right)\right]^{0.5} = 106$$

$$\dot{W}_s = \frac{\dot{W}}{\dot{W}_a/\dot{W}_s+1}$$

$$W_s = \frac{W}{W_a/W_s+1} \qquad \text{assuming equal time functions.}$$

$$= \frac{W}{106+1} \qquad = 0.00935\ W$$

$$= 1\%\quad W.$$

Where the stress pulse arises from the impulse delivered to the ground directly from the explosive materials, the wave front may have a spherical shape immediately under a surface burst,[4] e.g. point A in Fig. 1. This means that associated with the radial compressive stress there will be in elastic ground tangential tensile stresses equal to as much as half the magnitude of the compressive stresses. It is possible that under a dynamic stress pulse the static relation between radial and tangential stresses may not, owing to inertial effects, exactly apply.

Photoelastic studies have recently shown that the stress distribution around an opening resulting from a dynamic stress pulse can be determined by applying the normal static equations to the field stress that would exist at the center of the opening due to the stress pulse.[20] One important condition to this conclusion is that the stress pulse must have a significant rise time. However, theoretical studies indicate that the same conclusion applies for a pulse with a small rise time.[22]

If the static relations are used, the dynamic stress distribution around the opening can then be calculated. In Fig. 3c the effect on the opening from being engulfed by a stress pulse with a spherical front is shown. A large tensile zone is created in the roof. The maximum stresses shown here are about 12 kbar compression and 8 kbar tension for point A in Fig. 1. These would probably cause the opening to collapse, although the possible dynamic compression strength of 24 kbar would not be exceeded.

If point B in Fig. 1 is considered it is possible that the spherical wave front would not exist here. However, as it is a more conservative assumption than a plane wave front it may be used in design until more information is available. Making the same type of calculation as for point A a large tensile zone would be created in the walls. In this case, the compressive tangential stresses would not be severe but the tensile stresses would cause radial cracking in the walls.

Alternatively, at point B the most severe dynamic stresses might result from the air blast effects. Figure 1 shows the shock front that would propagate from the air blast into the ground.

The wave front, assuming ground with a constant seismic velocity, should be straight in a vertical plane but would be cylindrical in a horizontal plane. This cylindrical shape, in elastic ground, would then tend to give rise to horizontal tangential tensile stresses equal in magnitude to the radial compressive stress. Again this static relationship might not exactly apply for the dynamic case.

The stress distribution around an opening in this case will depend on whether the stress pulse impinges on the opening in a radial or in an axial direction. Assume for simplicity that the wave front makes a small angle with the horizontal, i.e. the air blast velocity is much larger than the seismic velocity. Then when the front strikes the opening in the radial direction, additional compressive stresses in the walls and tensile stresses in the roof will be created. Alternatively, if the horizontal, i.e. the air blast velocity is about equal to the seismic velocity, a radial pulse will tend to create tension in the walls and compression in the roof.

With the opening engulfed in the axial direction the concentration effects will be on the horizontal tensile stresses associated with the cylindrical shaped front. Hence, additional tension will be created in the roof and compression in the walls. In all these air blast cases the compressive stresses will not be severe for hard rocks but the tensile stresses are likely to cause considerable radial cracking and opening of joints.

It is assumed that any underground openings in rock that are likely to be subjected to nuclear blast effects will be deep enough so that any stresses arising from the surface Rayleigh waves would be insignificant. It has been shown that near the surface these effects might be important.[(23)] Also, for underground openings in rock the effect of the shear wave following the compression wave is not likely to produce in homogeneous ground conditions as severe as those associated with the compression wave.[(8)] However, where distinct weaknesses exist adjacent to an opening the shear wave effect should also be determined. The horizontal tension that might be created by the rapidly moving air blast[(59)] would only be expected where the shock front velocity is close to S-wave velocity.

The above calculation of the dynamic stress pulse was based on computed pressures existing in the crater[(5)] and assumed propagation functions. As it was assumed that there was no plastic zone or attenuation around the crater these figures may be upper limits. However, the magnitude of the calculated figure is very sensitive to the assumption regarding the correct exponent to be used for the variation of pressure with distance beyond the crater. The assumption of spherical attenuation is probably only valid for a zone within an angle of about 45% from the vertical.(5)

In addition, little information exists on the frequency that could be expected in the stress pulse. Considering the duration of the pressures that exist in the crater the pulse should have a very low frequency with possibly some high frequency components superimposed on this major wave. The assumption of a fairly large rise time in the pulse may or may not be valid. Such work as has been done, even in hard brittle rocks, suggests that this assumption is quite reasonable. In this case, besides the calculation of stress concentrations from static formulae being valid, the probability of tension slabbing occurring would (as shown in Fig. 4) be remote.

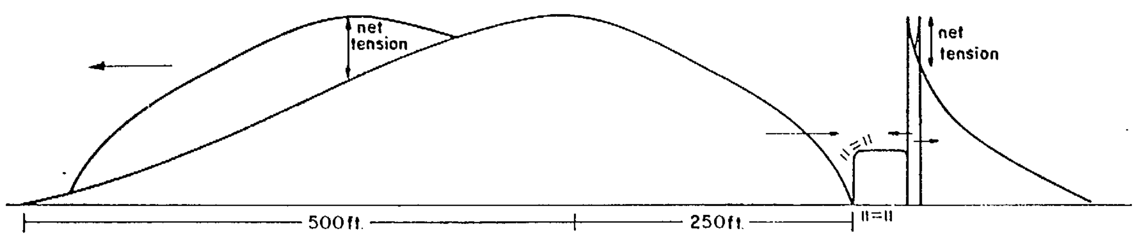

FIG. 4. The effect of pulse length on reflected tension.

The results of an extensive series of field tests to determine empirically the relations between damage to tunnels and the yield of an explosion, the burst of configuration, soil or rock type, and distance have been compiled.(1) It was found that there was a good correlation between the radius of the bomb crater and the distance to various types of damage produced in tunnels. However, the original work (15,24,25) on which these correlations mainly were based was done with TNT charges of relatively low magnitude. Consequently, the extrapolation of this information to large yield nuclear explosions is questionable as the shape of the stress pulse is likely to be different for the two different types of explosion (see Fig. 4), which could make a significant difference in the dynamic stress distribution around an underground opening. Also, considering the distribution of weaknesses within a geological material the size of openings should also be scaled in accordance with the similitude requirements of this property. It should be noted that the scaling of the size of opening thus should not be merely with the cube root of the yield.

Once the maximum stresses have been determined, it remains to compare these stresses with the strength of the rock. The determination of the actual strength of the rock, even under static loading, is not a simple matter.(29,27,28) Under dynamic conditions the strength for most rocks could be expected to be higher than under static loading. Some work has shown the dynamic strength to be as much as 12.25 times greater than that determined by static testing;[6] other work showed dynamic compressive strength to be 7 and tensile strength 25 times greater than static values;[9] whereas other strain measurements indicated that it might be only as much as 50% greater.[29] Work on concrete and shales also indicates significant increases in strength under dynamic loading.[30,31] However, there is little test data on competent rocks, and furthermore the techniques for determining such increases in strength have not yet been perfected. Again judgment based on as much information as can be obtained in testing program must be used to provide answers at the present time.

When numbers have been selected for maximum stresses and strength, it remains to be questioned whether ground will fail under stress concentrations. Considering the infinitesimal thickness of ground which is subjected to the maximum stress it is possible for minor deviations from perfect elasticity to significantly modify the distribution. In one case it seemed that the stress concentration required to produce failure was 10% greater than the loading required to produce failure on a normal sample[21] whereas in another case the theoretical stress concentration had to exceed the strength by 100–200% before failure occurred.[32] One explanation of these phenomena is the possibility of relaxation of surface tangential stresses. The actual occurrence of this phenomenon has been demonstrated.[33]

Where a series of openings exist side by side the stability of the resultant pillars might be of some concern. If the openings are close together the stress concentrations associated with a single opening become added to each other to produce great increases in average pillar stress.[34] Under these circumstances any weak planes in the ground can lead to failure with rockburst characteristics.[35]

However, in predicting the average pillar stress created by multiple openings some account should be taken of the transfer of load through shear into the outside abutments of the zone of excavation as has been observed by field measurements and photoelastic experiments.[39,37] A method has been determined for calculating this effect[38] which makes it possible to determine the order of magnitude of such a reduction and consequently helps in avoiding overdesign. For example, if a series of four tunnels connected by service drifts at their ends produced, in effect, a 75% extraction ratio and the length of the excavation area was half the depth below the ground surface, the average stress in the central pillar would be about 2.5 times rather than four times the vertical stress.

Another approach in predicting the effects of nuclear explosions on underground openings is to extrapolate directly empirical data which has been obtained from field experiments.

The main problem in scaling empirical data is to determine the functional relations between stress propagation and the yield of the bomb, the configuration of the blast with respect to the ground surface, the orientation of the tunnel with respect to the bomb burst, the distance from the burst and the properties of the rock.

Theories indicate, depending on the assumptions made, different functional relationships between these variables.(16,39) Analysis of empirical blast data for TNT in terms of exponential functional relationships has indicated a wide variation in the values of the exponents.(16,24,25,40,44) Consequently, the selection of the appropriate exponent is difficult. Alternatively, it is possible that a decay function relating stress and distance would be more suitable.(45,46)

A possible method of analysing this data is shown here. For installations that are above an angle of 45° to the vertical from the center of the explosion it has been suggested(47) and some measurements(9,48) confirm that the peak dynamic stress will vary somewhat as follows

$$6_f = K.W.^{0.5}/R^{1.5}$$

where 6_f = peak dynamic free field stress causing failure of rock around and opening;
K = parameter, assumed constant here;
W = yield of explosion;
R = distance to opening.

Hence

$$(6_f'/6_f'')=(K'/K'')(W'/W'')^{0.5}(R''/R')^{1.5}$$

$$R'' = R'(6_f'/6_f'')^{0.67}(W''/W')^{0.33} \text{ if } K' = K''.$$

Assume $6_f \alpha Qu$, the compression strength of the rock.
Thus

$$R'' = R'(Qu'/Qu'')^{0.67}(W''/W')^{0.33}$$

The Logan shot, 4.5 Kton, can be used as an example. A tunnel collapsed at a distance of 820 ft and discontinuous spalling occurred out to 1970 ft.(49,50) The compression strength of the rock in the area is about 6000 psi. Using the above expression and the calculation that only 1% of the surface burst kinetic energy is imparted to the ground, the equivalent distance to the sample case to which openings would collapse is:

$$R = 820(6000/30{,}000)^{0.67}(1000\times 0.01/4.5)^{0.33} = 366 \text{ ft.}$$

The distance within which openings would spall would be:

$$R = 1970(6000/30{,}000)^{0.67}(1000\times 0.01)^{0.33} = 870 \text{ ft.}$$

In the computations the constant K must account for the orientation of the stress which actually causes failure around the opening, the stress concentration effects of the opening, the direction of the pulse with respect to the axis of the tunnel, the direction of the pulse with respect to the surface of the ground, the length of the pulse and the possible variation of the pressure from the explosion around the perimeter of the crater or camoflet. These factors are not likely to be constant from one case to another. Whereas more variables have been included in this analysis than has been the case for most empirical extrapolations, merely listing those that have not been included points out some of the weaknesses in such empirical analyses. In any event, it is considered that the extrapolation of such data, owing to the radical difference in stress pulses that can be expected from TNT bursts, should be only from cases with nuclear explosives.

4. GROUND SUPPORT

It follows from the above discussion that a lining may be required to support loose ground that has been created by various tangential tensile stresses. If the compressive stresses were great enough to fail the rock then it is probable that the opening would collapse, which could not be prevented by any practicable lining. As mentioned above the probability of tension slabbing being effective seems to be remote for nuclear explosions.

Mass concrete might be considered for lining an opening; however, it has several disadvantages. If it is poured against the rock it will have an intimate, keyed contact. Consequently, the stress concentrations that have been calculated above would act, with a reduction of the order of 25% resulting from impedence mismatch in the concrete. As the concrete would probably be of lower strength than the rock itself no advantage would thus be gained. In fact, the concrete might fail where the rock would not. In addition, such a lining for large openings is fairly expensive.

The situation could be improved by placing a layer of soft or loose material between the concrete and the rock walls. With the resultant high impedence mismatch between the rock and the soft material little stress would be transmitted into the concrete from the stress pulse. Consequently, if the rock failed the concrete lining would still be intact to support the fractured ground. Furthermore, the soft filler material need only apply a low back pressure on the fractured rock (and conversely on the concrete lining) to establish a state of plastic equilibrium.[10] The main objection to such a scheme would be the very high expense.

The concrete might be strengthened, particularly in tension, by the addition of reinforcing steel. In certain circumstances and particularly for isolated zones of weaknesses, this is a practicable and competent lining. To use it for an entire underground installation, however, would make it a major item of expense.

Steel sets together with timber lagging might be considered. In this case provision would have to be made to prevent corrosion of the steel and rotting of the wood. The main objection again to some such system would be the very high expense involved in such a lining.

The remaining practicable method of lining a large underground opening is by the use of patterned rock bolting. It has been demonstrated that a mass of unconnected rods can be made into a beam by transverse bolting.[11] Thus it is reasonable to assume that the same thing can be done more easily with fractured rock. In other words, rock bolts under tension should not only inhibit failure of walls and roofs by keeping joints tight but should also knit a mass of failed, loose rock into a beam that can support itself.

By examining the various stress patterns that may develop around the openings the zones to which such rock bolting should be applied can be identified. It has been suggested that the ratio of length of bolt to spacing should not be less than 2 and that the ratio of spacing to rock fragment size should be less than 3.[11] These specifications are considered to be conservative, and it is thought that, particularly in walls, the length of bolts could be reduced (see Fig. 5).

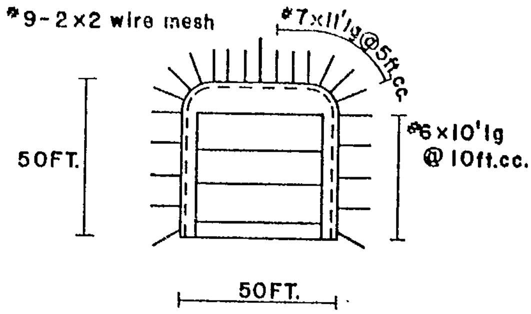

FIG. 5. Ground support system.

Combined with such a patterned rock bolt system it is important to include a wire mesh to support the loose rock that can develop between the bolts and in falling could cause considerable damage.

A rock bolt and mesh lining system has been found to be considerably cheaper than other methods of lining large openings. This lining need only be strong enough to support the beam of loose ground which might develop and to supply some small amount of back pressure. Then even if the ground beyond the rock bolts has been fractured it can remain stable under a condition of plastic equilibrium.[10]

Special treatment is required for special situations. For example, at the intersection of two openings it may not be possible to choke down the size of opening so that the diagonal span will not be greater than the normal chamber span.[51] In this case the static and dynamic stress distributions around the intersection should be examined and a special rock bolting system designed. For the cases where pillars, particularly at intersections, break well beyond the pay-line, the designed bolting system can be strengthened by splitting the spacing of the designed system with an emergency supply of longer bolts. Faults and weak bands of material should also be given special treatment; the analysis of these bolt requirements has already been postulated.[52]

In designing underground openings and their support systems, the traditional approach of the civil engineer in using factors against failure stresses should not be adopted. It is impossible to determine the precise strength of most rocks; indeed, we should probably expect such materials to have a variation in strength with some statistically determinable distribution about a mean value. Whereas we have some information that indicates that this variation applies to rocks loaded under static conditions, it is possible that it applies even more so when the loading is dynamic. In addition, for underground openings designed to resist the effects of nuclear explosions there is no certainty regarding the loading conditions. Again this is a matter which can be subjected to a probability analysis. Therefore, all elements of such an underground installation should be designed for some possibly arbitrarily* selected small probability of failure. At the moment it may be difficult to follow such a procedure rigorously throughout the entire design. However, it is thought to be useful to think in terms of this design theory so that advances can be made in the right direction, and overdesign and excessive expense can be minimized.

In the example shown in Fig. 3a the distribution of stresses occurring around the opening due simply to gravitational stresses would produce only one critical zone. This would be in the centre of the roof where a tension zone about 20 ft wide and a maximum of 10 ft deep would exist. Although the stresses would be low they might be sufficient over time to cause development of loose rock in this zone.

Should a horizontal orogenic stress considerably greater than the vertical gravitational stress exist in the ground a tension zone would be created in the walls. Again if such a zone were actually to exist it would probably result in the development of loose rock in this area.

In the cases of dynamic stress distribution where tensile stresses would exist at the wave front, high tangential stresses would occur in either the walls or the roof. These stresses would cause radial cracking and produce loose rock.

The rock bolt system must then be designed to support the loose ground that might result from the tensile zones created by the various cases of stress distribution. Thus it is considered that the prime function of the rock bolts should be to create a tight, prestressed masonry arch around the

*This design parameter should actually be determined by a marginal cost analysis whereby the marginal cost of the installation is related to the marginal cost of causing failure.[53]

opening. In this way any potentially loose ground would be stabilized, and in addition, the natural joints would be kept tight so that the expansion or weathering of weaker material in the joints or in veins would be inhibited.

To fulfil these functions the bolting should be placed as soon after excavation as possible. At great depths a period of time might be required so that the bolts would not be overstressed if tests showed significant elastic afterworking of rocks. The bolts are required to maintain their tension during the life of the installation, which is generally possible in competent rock by tightening them once or twice after the initial installation. Load cells can be installed on a selection of rock bolts to observe any variation of load with time. In addition, to ensure anchorage under blast conditions and to protect the steel against corrosion the bolts should be grouted. It can be easily appreciated that to create a continuous prestressed beam or plate a predetermined pattern of rock bolting must be installed regardless of the appearance of good ground.

The bolt diameter, length and spacing can be determined by calculating the weight of rock that might hang on one bolt using the ultimate strength of the bolt. The ratio of length to spacing should be about two and the bolt diameter should be of an economic size for drilling and installation.

The size of mesh that would be required can be analysed using arching theory as applied to granular materials.(54) For the bolting system shown in Fig. 5 the mesh required is calculated as follows:

$$w = \frac{\gamma b}{K \tan \phi}[1 - \exp(-Kn \tan \phi)]$$

where w = pressure on mesh,
γ = density of fractured rock; assume 150 lb/ft^3,
b = spacing of bolts,
K = ratio of horizontal to vertical stress in the fractured rock; assume 0.25,
ϕ = angle of internal friction of fractured rock; assume 45°,
n = ratio of depth of bolts to spacing,

$$w = \frac{5 \times 150}{0.25 \tan 45}[1 - \exp(-0.25 \times 2 \tan 45)] = 1060 \text{ lb/ft}^2.$$

Assume the mesh sags into a parabolic shape with a maximum sag of 0.5 ft; analyse a two-dimensional case, and use galvanized chain link mesh with an ultimate strength of 95,000 psi. The area required is

$$A = \frac{1060 \times 5}{2}\sqrt{1 + \frac{5^2}{16 \times 0.5^2}} \div 95{,}000 = 0.075 \text{ in.}^2/LF$$

Number 9, 2 in. by 2 in., chain link mesh with 0.104 in.^{2}/LF (square inches per lineal foot) would satisfy this requirement.

5. GROUND SHOCK

Underground openings may contain instruments, equipment and structures. These elements would be subjected to accelerations arising from the movement of the ground as the stress pulse engulfs the opening. As these accelerations might be sufficient to damage structures and equipment it is important to be able to predict the nature of the ground motion. Owing to the high stress levels in these waves classical seismology was not applicable to this problem.

A method derived by Newmark[55] can be used for determining the maximum components of ground motion some distance away from the crater if the major stress pulse arises from the impulse applied to the ground by the materials in the explosion. A calculation using this method for the case described in Section 3, with the main chamber 1500 ft from the explosion and the air blast peak overpressure 1000 psi, is as follows:

$$\begin{aligned} d_v &= 1.9\, W^{0.83} R^{-1.5} \\ &= 1.9 \times 1 \times 1.5^{-1.5} && = 1.03 \text{ in.} \\ a_v &= 0.36\, W^{0.83} R^{-3.5} C_L^2 \\ &= 0.36 \times 1 \times 1.5^{-3.5} \times 15^2 && = 19.5 \text{ g} \end{aligned}$$

where d_v = vertical free field displacement in inches;
W = yield of explosion in Mton;
R = distance in kft;
a_v = vertical free field acceleration in g;
C_L = seismic velocity in kft/sec.

These equations were derived using both theory and empirical data. The variation of scaled acceleration with distance for an underground nuclear explosion provided the basic functional relationships. Then by assuming a velocity–time wave shape the free field particle velocity and displacement were obtained. The resultant equations were then converted so that they could be used for surface bursts by assuming the magnitude of the partition of mechanical energy between the air and ground in a surface blast. These equations are intended to be used only in the range corresponding to air blast overpressures between 100 and 600 psi and were derived solely for homogeneous granite.

The velocity wave shape assumed in this derivation is shown in Fig. 6. Both the rise time and duration were assumed to vary directly with the distance and inversely with the seismic velocity. Some empirical work substantiates this assumption,[15,41,45,50] whereas other work does not.[9,24,25,29,40] One would expect that a visco-elastic material would cause rise times to increase with distance; however, a locking material, or one with an increasing modulus of deformation with increasing stress, should produce a shock front. Also, duration should vary with yield as has been shown in TNT experiments.[9]

The derived acceleration from the velocity curve can be compared with an actual acceleration curve as shown in Fig. 6d.

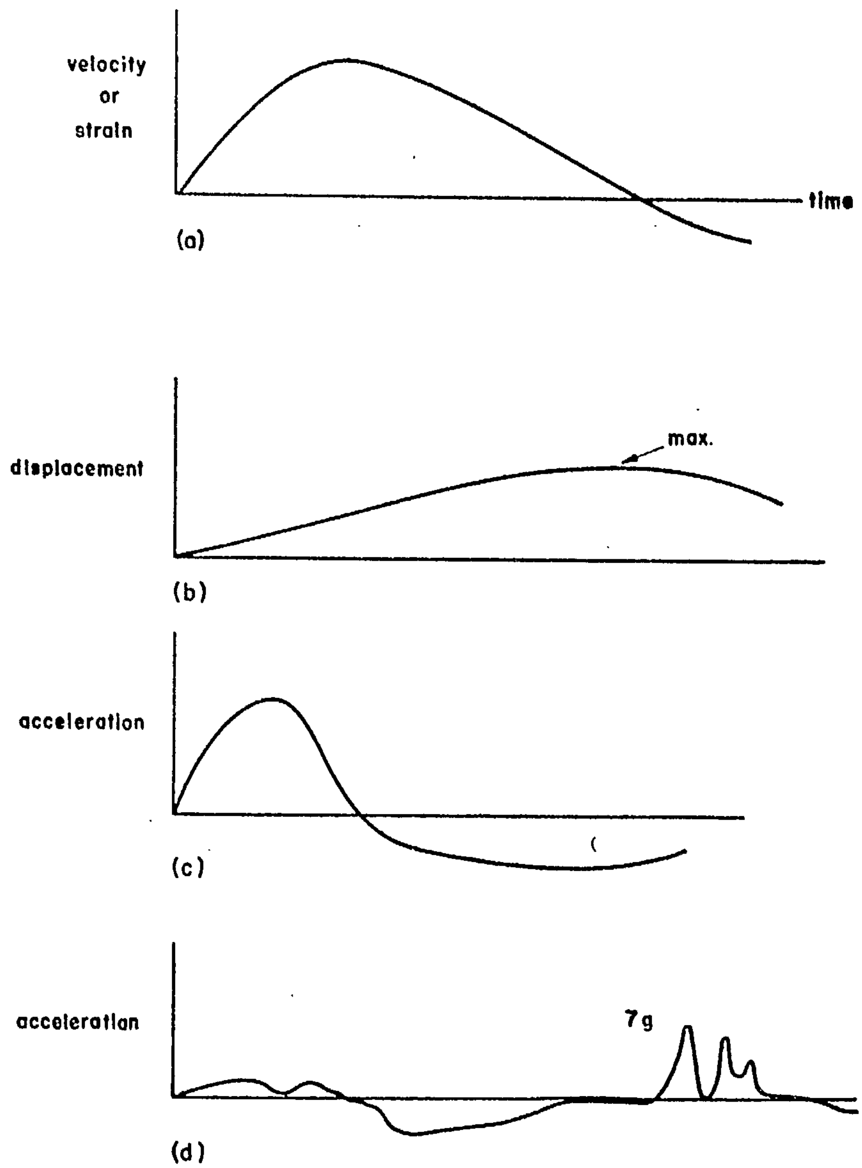

FIG. 6. (a) (b) (c) Assumed wave shapes for determining ground motion; (d) a measured wave at 30 ft depth.

A method established by Sauer[5] can also be used for computing the same ground motion. This method is based on exhaustive analysis on ground motion measurements from nuclear explosions. The corresponding calculations, although valid only for a depth of 10 ft, follow:

$$\begin{aligned} d_v &= 7200\ W/(S \cdot C_L \cdot R^2) \\ &= 7200 \times 1/2.68 \times 15 \times 1.5^2 \qquad &= 7.95 \text{ in.} \\ a_v &= 200\ W^{0.67}/(C_L\ .\ R^2) \\ &= 200 \times 1/15 \times 1.5^2 \qquad &= 5.92 \text{ g} \end{aligned}$$

where s = the specific gravity of the rock.
The attenuation function of this motion with depth is not presently available for circulation.

The free field ground motion that would occur from the shock pressures in the air acting on the ground surface can be calculated from another set of equations derived by Newmark.[56] An example of these equations follows:

$$\begin{aligned} d_v &= 10(P_{SO}/100)^{0.4} W^{0.33}/C_L \\ &= 10 \times 10^{0.4} \times 1/15 \qquad &= 1.67 \text{ in.} \\ a_v &= \frac{2.58 P_{SO}}{z} 1 \bigg/ \left(1 + \frac{z(P_{SO}/100)^{0.6}}{300 W^{0.33}}\right) \\ &= \frac{2.58 \times 1000}{600} 1 \bigg/ \left(1 + \frac{600(10)^{0.6}}{300 \times 1}\right) \qquad &= 0.48 \text{ g} \end{aligned}$$

where P_{SO} = air blast peak overpressure in psi,
z = depth to acceleration in feet.

These equations are based on the theoretical relations that exist in the case of a long rod between the material properties and the particle motion due to a stress pulse. The coefficients have been modified by analysing empirical data. In addition, the attenuation of velocity and acceleration with depth is based on the geometrical or spherical attenuation of the expanding wave front.

Again an alternate method derived by Sauer[5] can be used for calculating the ground motion due to the air pressures acting on the surface. These equations are also based on an analysis of a large amount of empirical information.

$$\begin{aligned} \mathrm{d}_v &= 0.24.I.\ P_{SO}{}^{0.25}/(S.C_L) \\ &= 0.24 \times 46 \times 1000^{02.5}/2.68 \times 15 \qquad &= 1.55 \text{ in.} \\ a_v &= 0.34\ P_{SO}/C_L \\ &= 0.34 \times 1000/15 \qquad &= 22.7 \text{ g} \end{aligned}$$

where I = positive phase impulse in air blast, in psi/sec. Again the attenuating effects of depth are not available for circulation.

Figure 7 shows the envelopes of free field motion predicted for the motion at 600 ft below the surface resulting from directly induced motion and from air blast pressure.

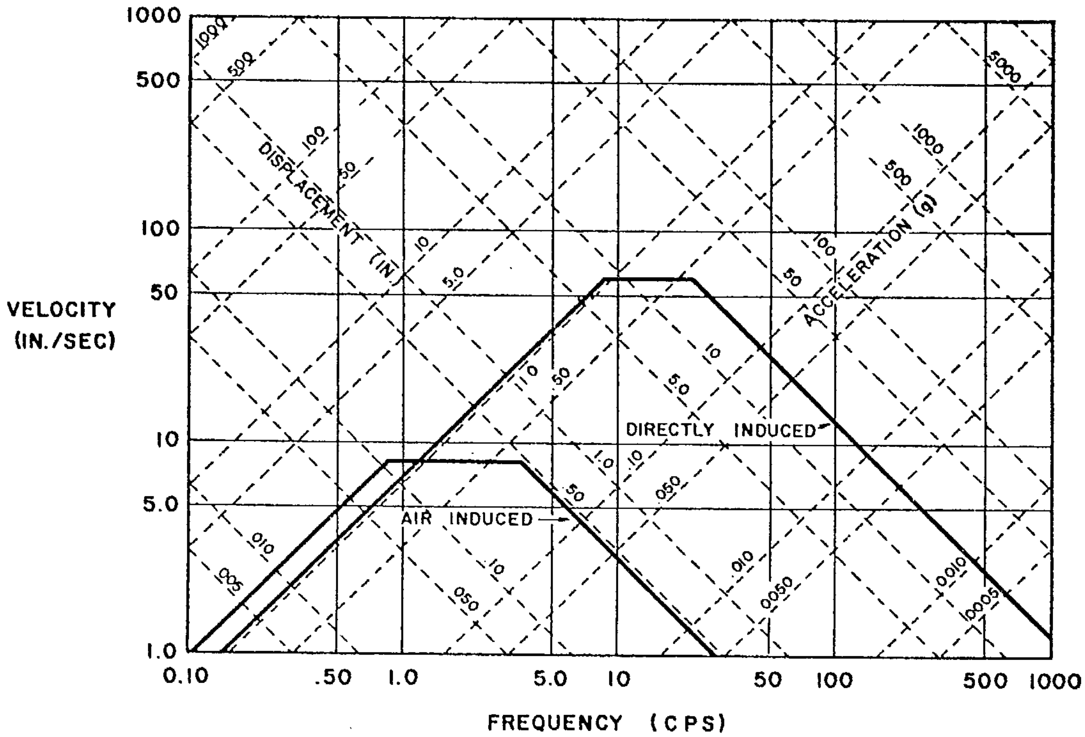

FIG. 7. Envelopes of ground motion at 600 ft depth from air blast and from direct impulse for 1 MT at 1500 ft in hard granite.

A simple approach and one that could possibly be developed to provide a more satisfactory theoretical framework is to examine the propagation of the strain pulse from the crater. For the 1 Mton burst one might derive the following equation:(5)

$$\sigma_r = 5.6 \times 10^3/R^{1.5}$$

where σ_r = peak radial stress in kbars;

R = distance in m.

Hence $e_r = 3.6 \times 10^3/(R^{1.5}E)$,

where e_r = radial strain,

E = modulus of deformation.

Then $v_r = e_r \times C_L$

where v_r = radial free field particle velocity.

It would then be necessary to know or assume the shape of the velocity–time pulse to integrate for displacement and differentiate for acceleration.

In computing the stress pulse characteristics it is necessary to know the seismic velocity of the ground. As a practical matter, this property is difficult to determine.

The seismic velocity can be measured by the normal geophysical methods. However, the stress pulse associated with these methods is very low and consequently may not be applicable to a stress pulse of a much larger magnitude. If the rock were brittle and perfectly elastic then the seismic velocity should be the same for both cases; however, most rock, particularly under field conditions, will produce stress–strain curves with some curvature, e.g. jointing tends to produce a stress–strain curve with an increasing modulus whereas the rheological nature of many rocks tends to produce stress–strain curves with a decreasing modulus depending on the duration of stress.

A laboratory vibration or sonic method of determining seismic velocity would seem to be less satisfactory than the field method owing to the absence of any joint systems that might affect the actual field conditions; although, under high stress pulses these might not be of great significance.

A static compression test in the laboratory can be used to determine the modulus of deformation at high stress levels. Then, with the determination at the same time of Poisson's ratio, it is possible to calculate the corresponding seismic velocity. This number again might be too high owing to the absence of joints in the laboratory sample or it might be too low owing to the loading being applied statically. At the moment there is no satisfactory solution to this problem, and judgment must be used in each case taking into account all the measurable material and geological factors.

The actual response of the structural elements in the underground opening will depend upon their dynamic characteristics. Magnification of the free field motion can occur if some resonance occurs between the structural elements and the frequency of the ground motion. However, this magnification is limited owing to the ground motion probably having at most only one cycle of velocity or a half a cycle of displacement.[9,48] Methods have been established for estimating the response in one direction at a time of simple structural elements.[56] Suffice to say here that besides providing for the dynamic structural stresses, relative displacement between a structure and the chamber makes "rattle space" necessary as shown in Fig. 5.

Owing to the severe motion that these structural elements may be subjected to, the isolation of the structures from the rock may be required. One cheap method that was examined included the use of a sand bed on which the structure would be placed. The isolation characteristics of such a sand bed are shown in Fig. 8.

It can be seen that for footings of reasonable sizes it is not possible by increasing the thickness of the sand bed to obtain a natural foundation frequency of less than 3 c/s. Furthermore, even by increasing the size of the footings it was not possible to obtain a natural foundation frequency of less than about 2 c/s. In other words, the lower frequency motion would pass through the bed. As it is generally necessary to obtain an isolation system with a frequency of less than 1 c/s this system would not normally be useful.

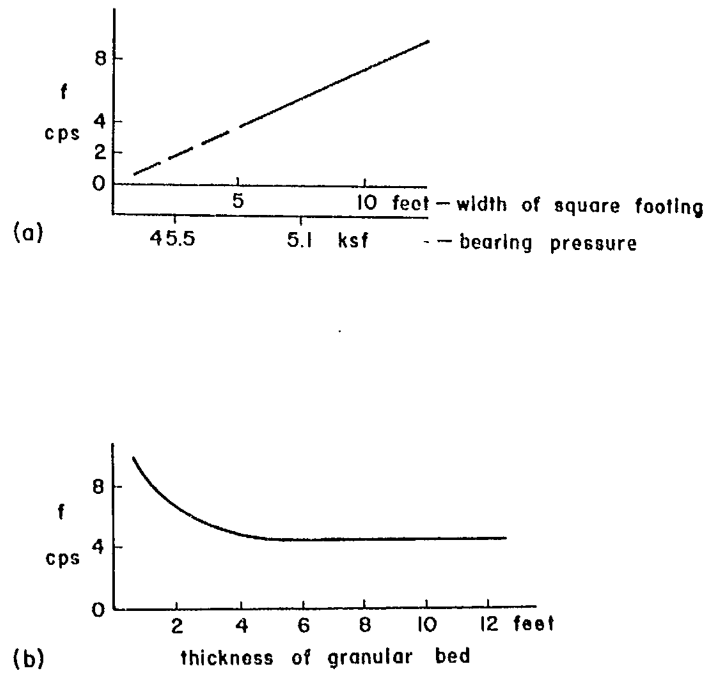

FIG. 8. Shock isolation properties of a bed of sand or broken rock.

Suggestions have been made in the past that shock isolation should be obtained by blasting the floor thereby creating a granular bed of material. However, the above analysis for the sand bed would also apply to this system indicating that the low frequency motion would still be transmitted through such a layer of broken ground. At the moment it seems necessary to use either coil springs or pendulums for isolating structures from ground motion.

For some structures the possibility of reciprocating motion would be important.[57] It would seem that at most two cycles of velocity with a damped roughly sinusoidal shape might be possible; however, this might not occur until relatively far away from the explosion.[15,40,41,43,48] Nevertheless, in the face of our present ignorance, together with the possibility of several explosions occurring, this factor should be included in the design of the installation.

6. SERVICE ENTRANCES AND EXITS

Most underground installations require openings extending to the surface for various service requirements. Access is required for personnel and supplies. Air may be required for personnel and power plant. Information must be received and communication ensured with other units. Cooling water may be required. The disposal of exhaust gases, sewage and hot water may be necessary. Figure 9 is a schematic diagram of the possible requirements of such an installation.

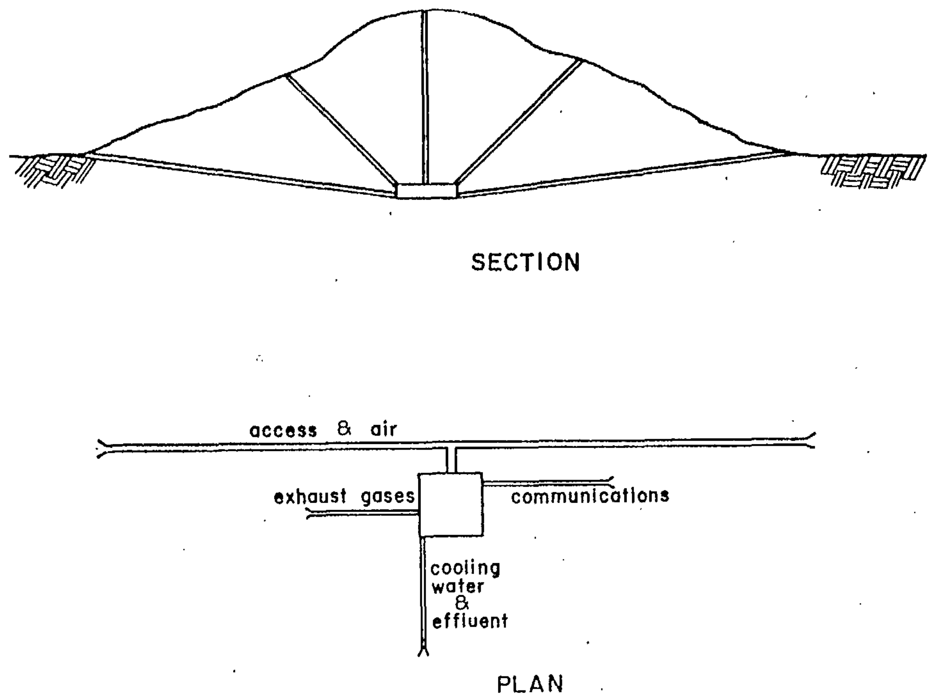

FIG. 9. Typical layout of an underground installation.

It can be seen that such adits and raises may collapse under the effects of the dynamic stress pulse produced by a nearby surface explosion. Alternatively, the portals may be blocked with debris fallout from the stem of the burst, throwout from the crater, or local scour from the blast winds. In addition, such openings permit the entry to the installation of air blast, high temperature gases and radioactive particles.

The appraisal of the ground stress effects requires the same type of analyses as were discussed in Section 3 for the main chambers. In addition, high temperature gradients around openings such as diesel exhaust raises may also produce spalling.

The alternatives for reinforcing the ground against failure are the same as those discussed in Section 4 for the main chambers. If these passages pass through the same type of ground as the main chambers the support problem is less difficult owing to the smaller size but the requirements must be resolved on a probability basis. However, the likelihood of these passages having to go through weaker ground is very probable on most sites. Consequently, additional support may be required, or the provision of alternate routes for the service may be necessary. This decision is made on the basis of the cost of equal alternatives.

The greatest intensity of debris will occur at the lip of the crater. From analysing actual explosions it has been deduced that the width of the lip will be approximately equal to the radius of the crater and the maximum thickness of the debris in the lip will be approximately one quarter of the depth of the crater.(1) This means that the major part of the material removed by the explosion from the crater is placed on the lip. Any portal in this area, even if the ground did not fail, would thus be blocked with debris. If the probability of this occurring is significant then alternate routes must be provided.

Some of the debris from the crater will be deposited beyond the crater lip. Empirical studies have shown that the following relationship applies to one particular site where the surface material was a caliche:(58)

$$t = 5.1 W^{0.29}(R_c/R)^{2.7}$$

where t = thickness of debris in cm,
W = yield of TNT explosion in lb,
R_c = radius of crater,
R = distance to debris of thickness t.

For the case described in Section 3 and assuming the above empirical equation could be extrapolated to nuclear explosions and other sites the calculation would be as follows:

$$t = 5.1(0.5 \times 10^6 \times 2000)^{0.29}(520/1500)^{2.7} = 117.4 \text{ cm} = 46.3 \text{ in.}$$

This calculation takes into account that the yield of a nuclear explosion produces about half the mechanical energy of a TNT explosion of the same yield. The resultant thickness would be an average with local variations. Information is required to determine if these functional relations exist for ground other than that tested.

In addition to the crater throwout and the fallout from the stem of the blast, the scour of wind action can be significant. At a distance of 3500 ft from a 1 Mton blast it could be expected that a 6 ft or 10 ton boulder would be moved 50–100 ft, whereas at a distance of 1500 ft this boulder would be moved 300–400 ft. Consequently, it would be desirable to design portals and collars to avoid being plugged by such debris. Owing to the large amount of drag that would be applied to any portal structure elevated above the ground surface this is generally very difficult to do. Alternatively, duplicate or multiple exits can be provided. The amount of money spent on this aspect must be related to the probability of the occurrence.

Blowing plugged exits open with water or air under pressure has been considered to provide for at least the passage of water or gas. Analyses have shown that the pressures or heads required for this action, although not unreasonable, tend to be rather high.

Protection against the effects of air blast, hot gases and radioactive particles is relatively simple and normally does not require any rock mechanics analyses. Where there is no need for portals to remain open an interesting design problem is the provision of a self-collapsing entrance to eliminate air blast. This can usually be achieved by providing an L- or S-shaped entrance which will collapse under the air blast pressures acting on top of the entrance structure before the air blast has time to travel the longer distance around the S and penetrate the tunnel. Multi-plate pipe with some fill can be used for such a portal.

7. RESEARCH REQUIREMENTS

Much has been learned during the past few years on the rock mechanics aspects of designing underground openings to resist dynamic stresses. However, many important features must still be determined on the basis of engineering judgment rather than scientifically determined facts. The following list indicates the topics on which rock mechanics research would be most helpful.

(a) In view of the ability to calculate the pressures that exist in the hot gases of the explosion it should be possible to predict crater dimensions in various types of ground by determining their strength parameters and using bearing capacity theory. There is a moderate amount of empirical crater data to substantiate or modify such theoretical work. The resultant equations would be very useful in analysing the effects of explosions very close to underground openings. In addition, it would supplement studies on debris distribution.

(b) A complete theoretical framework for relating ground motion and stress to the yield of the explosion and the ground properties is needed. There are several elements of research work that would be required for such a framework. The partition of energy between various types of radiation and kinetic energy and the division of the kinetic energy between the ground and the air for surface and subsurface bursts should be established. Then the energy absorption in causing ground to fail or yield plastically adjacent to the crater needs to be studied. At the same time the current theoretical work on the transmission of the pressure pulse through the various types of plastic zones that can exist around the crater should be studied experimentally.

(c) Much more information is required on the transmission of a high intensity stress pulse in various types of idealized, competent ground. The prediction of even minimum limits on the attenuation of the maximum stress with depth and with horizontal distance is very important. The change in shape of pulse with distance and its consequent effect on the ground motion components of displacement, velocity and acceleration is also important. Finally, the relations between radial and tangential motion and stress warrants more attention.

(d) More research work needs to be done on the strength of brittle materials, particularly under dynamic loading. This applies to both compressive and tensile stresses. In other words, more careful testing should be done with the ultimate objective of establishing a valid strength theory. In addition, the effect of thermal shock should receive some attention.

At the same time, studies should be carried out to determine if rock fails as a result of stress concentrations. Part of the study should be in the field to determine if stress concentrations exist as they are theoretically calculated. The other part of the study should be to determine the effect of a rapidly varying stress intensity such as is presumed to exist as a result of stress concentrations. This work is likely to require an examination of the effects on the distribution of stress around an opening of the rheological properties that exist in many rocks.

In addition, whereas current studies are leading to solutions of ground motion in idealized visco-elastic materials, practically no information as yet exists on the rheological properties of actual rocks—nor have the testing methods yet been standardized to obtain this information. The initial steps, at least, required to rectify these deficiencies are quite clear.

(e) Most of the empirical data that has been used to predict damaging effects from explosions has been obtained from TNT bursts. The reasons for this information not being applicable to nuclear explosions has been mentioned above. Consequently, every opportunity should be taken to build up sufficient information from nuclear explosions to substantiate or modify the existing empirical damage equations.

(f) More research work is required on the mechanics of rock bolting. The creation of masonry beams from jointed blocks by transverse rock bolting could be easily studied. The results of such studies would be of immediate benefit.

(g) In view of the need for inexpensive shock isolation for installations that may be subjected to severe ground motion, some research work should be initiated to study the relationships described in Fig. 8. It is possible that a sand bed or a floor of broken rock could be effective for some purposes. For example, even if such a bed transmitted the low frequency motion and hence produced large displacements, the filtering of the high acceleration motion might be quite useful.

(h) It would not be unusual for the service entrances and exits as shown in Fig. 9 to cost more than the main chambers of the installation. For this reason, some research into construction or mining methods for providing passages for such services might be profitable. Of course, for those not in the field of rock mechanics the incentive exists to achieve these services by means other than passing materials or signals through physical openings in the rock.

(i) More research work needs to be done on the distribution of debris associated with a nuclear explosion. The improvement in analysing this effect requires more work on the rock mechanics aspect but probably requires an equal amount of work on the probability aspect. The need for this information is quite urgent.

Acknowledgment—This paper is based largely on work done for the Royal Canadian Air Force in establishing design criteria for their SAGE installation.

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DISCUSSION OF D. F. COATES'S PAPER

R. H. CARLSON: It must be emphasized that the debris thickness relationship (page 557) was developed to fit data resulting from 5-ton TNT surface burst hemispheres. There are known discrepancies in the thicknesses predicted when this expression is extrapolated to nuclear yields using conventional efficiency or corrective factors. In cases where we have to estimate debris thickness surrounding a nuclear crater we use half of the thickness which results when the total nuclear yield is substituted for W in the relationship

$$t = \frac{5{\cdot}1\,W^{0,29}}{\alpha^{2,7}}$$

This one-half factor is merely an estimate based on crater dimension ratios between TNT and nuclear craters which have been observed for desert alluvium at the Nevada Test Site. When TNT crater dimensions and depths are extrapolated to 1·2 KT yields the linear crater dimensions are always greater than observed for the nuclear shot. Thus we have assumed a ratio for linear dimensions of 2 to 1 for HE to nuclear and therefore we divide the thickness predicted by the HE expression by 2.

There are numerous ways of looking at this thickness question from an energy partitioning or an efficiency standpoint. We know now that several of the assumptions which are implicit with the use of our HE debris thickness relationship to predict debris from nuclear craters are not valid. For example, we know now that the thickness profile is a function of yield even for HE craters. That is, for greater HE yields the mass of ejecta is observed at closer scaled distances from ground zero. We hope to develop a better expression for the prediction of nuclear ejecta thickness in the near future.

J. J. REED: Don't you think that creep may reduce the efficiency of bolt anchorage?

D. F. COATES: Yes, it is possible although I consider creep to be unlikely in a granite rock. I would also add that bolts should be grouted to protect against corrosion.

H. L. HARTMAN: A recent paper by Robert Stefanko* from Pennsylvania State University discusses methods of effectively anchoring roof bolts and evaluating bolt anchorage in almost any rock including shales.

D. ROSTOKER: It is possible that a hard granite may deform to a greater degree than some softer materials. Granites may be subjected to rather high natural stresses and are known to exhibit creep-like deformations.

J. J. REED: I would like to stress the importance of *in situ* determination of the properties of the rock. Values taken from tables can be very misleading. Control tests should be carried out throughout construction.

* R. STEFANKO. New look at long-term anchorage: Key to roof-bolt efficiency. *Mining Engineering*, **14**, No. 5, 55–58, May 1962.