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HIGHWAY RESEARCH RECORD

Number 135

Rock Mechanics

4 Reports

HIGHWAY RESEARCH BOARD

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Subject Classification

61 Exploration-Classification (Soils)
63 Mechanics (Earth Mass)
62 Foundations (Soils)
33 Construction

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Foreword

The geologist has long defined as rock all compositions of the earth's exterior in and on which man builds. Much of the knowledge obtained has been qualitative in nature. The engineer, in need of knowledge stated in quantitative terms, considered the derivations of rock as soils and proceeded to determine, through experiment and theory, quantitative values that described soil behavior. Thus, from the early studies on earth pressure (Coloumb 1776, Poncelot 1840, Rankine 1857), knowledge of the behavior emerged from that of a qualitative nature to one wherein behavior could be expressed mathematically. The publishing of his book "Erdbaumechanik" by Karl Terzaghi heralded the beginning of Soil Mechanics, the new science concerning the soil portion of the earth's crust.

Increasingly, present constructions reach depths below the soil mantle, some to depths of 200 to 300 feet and into partially consolidated or consolidated earth deposits. Here, as once was true for soils, much of our knowledge, even now, is of a qualitative nature. The engineer, closely cooperating with the geologist, is seeking quantitative data from which to predict rock behavior as related to his constructions. Here again, as a result, is emerging a new science which we term Rock Mechanics—which now is in its early state of development.

This RECORD contains four papers. Each of these add to our knowledge of one or more of the many segments of this new science that is of practical interest to those concerned with construction in, of, or on rock.

Emery presents a simple concept of rock as composed of "grains and glue" and how adjustments due to strains are taken up by the grains-and-glue makeup of rock. He presents equations for three rheological models and discusses how they relate the behavior of rock. He also points out some of the problems of highway design due to rock conditions and gives simple tests that can provide data to enable more rational design in rock. This paper should interest both the researcher and the practicing engineer.

The paper by Long, Merrill, and Wisecarver describes instruments and procedures that represent the beginning of a scientific effort to determine, by experiments, quantitative values associated with jointing, bedding, faulting, slope geometry, ground water, and the effect of overblasting on the stability of a rock slope. This paper should interest teachers, researchers, and practicing engineers, as well as those whose specialty is rock mechanics.

Grosvenor and Abel present data on load and convergence measurements in tunnels to determine the relationships between design load and loads imposed, the zone of influence around the tunnel, the variation in strain outward from the tunnel walls, and the rate of decrease of overall radial ground strain rates with time. The paper should interest engineering geologists, specialists in rock mechanics, designers and constructors.

A question frequently asked by engineers responsible for design concerns potential blast damage to nearby structures—the structures ranging from large bridges to family residences. Devine presents data on seismic particle velocity, distance to structure and maximum charge weight per delay, and their relation to the degree of damage caused by vibrations set up by blasts. His paper should be of interest to construction engineers, whether employed by the constructing agency or the builder.

Contents

The Strain in Rocks in Relation to Highway Design

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•IN HIGHWAY construction rock is often the foundation on which the highway is built. In the case of rock cut or a notch in a mountain slope the walls may also be rock. A tunnel may be completely in rock. The bed of the highway is often broken rock, as is the aggregate in the surfacing material.

To the designer, stability problems are involved in the site selection and in the rocks left in place after the cut or tunnel excavation is completed. There is a further problem in the stability of the rock fill to be used and yet another problem in the permanence of the aggregate surfacing, bonded or otherwise.

Another aspect often mistakenly left to the contractor is the use of the rock properties at the point of concern when planning the blasting patterns and the sequence of operations.

This paper will deal with some of the concepts of rock mechanics and their possible importance in highway design and construction.

ROCKS

A simple but useful concept of rock is that it is a granular material consisting only of grains and glue. There is nothing else involved. The grains may be small polycrystalline aggregate from a nucleation process or from a chemical precipitate, or they may be small fragments of other rocks. The glue may be ferruginous, calcareous, argillaceous, or siliceous material which cements the grains, or, in the case of an intergrowth of crystals, the outer layers of the crystals which are in contact with each other may be considered to act as a glue. Water may be a glue in a clay or a grain in a crystal structure. Air or other fluids or gases are grains if they occupy positions within a granular mass. Such grains are functional materials in the rocks, often chemically as well as mechanically. Most rocks have the grains arranged in some packing pattern and therefore have directional properties.

Rock may be described, then, as a granular aeolotropic heterogeneous technical substance which occurs naturally and which is composed of grains of varied polycrystalline or noncrystalline materials which are cemented together either by a glue or by a mechanical bond, but ultimately by atomic, ionic or molecular bonds within the grains and the glue and at every interface of bonding (1).

Primitive Strains

All technical materials if subjected to a change in load or a force of any kind tend to adjust to the force. Such adjustment may be a movement from one place to another, called a translation, or it may be a reorientation called a rotation, or there may be a change in shape or size as in an extension or compression of a material.

In rock all the adjustment must be taken by the grains and glue. The way in which adjustment occurs will depend on the material properties of the various grains and glues and will also depend on their packing pattern and on the time involved. Other papers (2, 3) have dealt with this and the concepts will be only summarized here. In general, permanent irrecoverable movement occurs as well as elastically recoverable movement.

Paper sponsored by Committee on Soil and Rock Properties and presented at the 45th Annual Meeting.

The amount of each will depend on the material properties, the time involved, the packing pattern and the kind of force field. The rate at which such a material can adjust without cracking will also depend on the material properties.

Rheology

Some simple rheological models may be useful here. No real material is perfect and all materials combine in some ratio the three characteristics of elasticity, fluidity and plasticity given by the rheological equations for perfect substances as follows:

1. $\sigma = E\epsilon$, the equation for a perfectly elastic Hooke solid;
2. $\sigma = \eta \frac{d\epsilon}{dt}$, the equation for a perfectly fluid Newtonian liquid; and
3. $\sigma = \phi$, the equation for perfectly plastic flow in St. Venant solids.

where

σ = unit stress;
ϵ = unit strain;
$\frac{d\epsilon}{dt}$ = rate of strain;
E = Young's modulus;
η = coefficent of viscosity;
ϕ = yield stress;
t = a parameter representing time;
ϵ_0 = unit strain at $t = 0$;
σ_0 = unit stress at $t = 0$; and
e = Naperian base = 2. 7183.

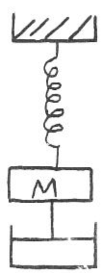

Figure 1. Kelvin solid.

Then for a substance below the short time yield stress

$$\sigma = E\epsilon + \eta\frac{d\epsilon}{dt}$$

represented in the spring, mass, and dashpot arrangement in Figure 1. This is a Kelvin solid. Here the full elastic strain is not instantaneous but subject to time lag as shown by the solution to the equation

$$\epsilon = e^{\frac{-E}{\eta}t}\left[\epsilon_0 + \frac{1}{\eta}\int_0^t \sigma e^{\frac{E}{\eta}t}\, dt\right]$$

if σ is constant,

$$\epsilon = \frac{\sigma}{E} + e^{\frac{-E}{\eta}t}\left[\epsilon_0 - \frac{\sigma}{E}\right]$$

and if $\sigma = 0$

$$\epsilon = \frac{\sigma}{E}\left[1 - e^{\frac{-E}{\eta}t}\right]$$

The elastic strain caused by a stress will reach a maximum only in infinite time and if the load is removed infinite time will be required for complete recovery of the elastic strain.

Also

$$\frac{d\epsilon}{dt} = \frac{\sigma}{\eta} + \frac{1}{E}\frac{d\sigma}{dt}$$

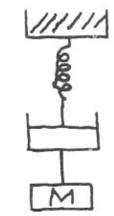

Figure 2. Maxwell liquid.

represented in Figure 2. This is a Maxwell liquid which has the solution

$$\sigma = e^{\frac{-E}{\eta}t}\left[\sigma_0 + \int_0^t E\, e^{\frac{E}{\eta}t}\frac{d\epsilon}{dt}\, dt\right]$$

Here the rate of strain is a function of the total stress combined with the rate of stress application. Given any finite time to flow, the reaction includes flow. Given no time to flow, the reaction is elastic. The stress will vanish only in infinite time.

Given the Kelvin solid model, consider a local application of load such that a local finite mass of the material strains but influences the background mass of which it is a part to strain in the same direction at a rate proportional to the strain in the finite mass (Fig. 3). Then the finite mass will move a distance x while the background mass moves a distance y to adjust to the first movement such that

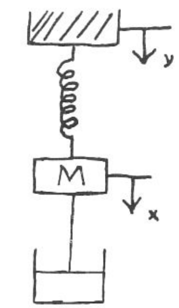

Figure 3. Kelvin solid with load.

$$\frac{dy}{dt} = -r\,x$$

Given

$$\sigma = E\,\epsilon + \eta\frac{d\epsilon}{dt}$$

this can be rewritten to conform to the mass-acceleration formula for such a system so that

$$F = E(x-y) + \eta\frac{dx}{dt}$$

and

$$M\frac{d^2x}{dt^2} + \eta\frac{dx}{dt} + E(x-y) = 0$$

then

$$M\frac{d^3x}{dt^3} + \eta\frac{d^2x}{dt^2} + E\frac{dx}{dt} - E\frac{dy}{dt} = 0$$

therefore

$$M\frac{d^3x}{dt^3} + \eta\frac{d^2x}{dt^2} + E\frac{dx}{dt} + E r x = 0$$

The characteristic equation is

$$\alpha^3 + \frac{\eta}{M}\alpha^2 + \frac{E}{M}\alpha + \frac{E\,r}{M} = 0$$

Let the roots be λ_1, λ_2, λ_3, or λ, $u - iv$, $u + iv$. For a cubic

$$\lambda + 2u = -A = -\frac{\eta}{M}$$

$$2\lambda u + u^2 + v^2 = B = \frac{E}{M}$$

$$\lambda(u^2 + v^2) = -C = \frac{-Er}{M}$$

The solution is $x = C_1 e^{\lambda t} + e^{\mu t}(C_2 \cos vt + C_3 \sin vt)$.

For stability both λ and μ must be zero or negative. Therefore A, B and C must all be positive. The Hurwic criterion for a stable third order system requires that $AB \geq C$, whence

$$r \leq \frac{\eta}{M} \text{ for equilibrium.}$$

Therefore when a load is applied (or removed) locally with respect to any volume of material the local deformation will propagate into the background volume and a progressive adjustment of the system will occur. Such adjustment will be acceptable or destructive depending on the value of r and on the properties of the material concerned.

It is known from tests that, in rocks, η decreases as the stress increases (4). For a given mass r must decrease when the stress is increased if the system is to reach equilibrium without destruction. This will depend to some extent on the shape of the mass as well as on the magnitude of the mass.

A fully restrained mass can accept a very large load and so develop a very low viscosity which results in a low r. The material can then adjust to a large deformation x without decoupling. For example, encapsulated water can support large loads. Triaxial compression tests are based on the use of a uniform shell of restraining pressure around the material undergoing test. The difference between the stress of restraint and the stress of the uniaxial applied causes eventual failure but the total load and the restraint are both so high that the viscosity of the material is low. Flow occurs before fracture.

When the restraint is small as in the case of the lateral surface of a sample in an ordinary compression test, η is large, r is large and the characteristic rate dy/dt will require failure to occur for a relatively small strain x.

Therefore, each material in a given transient state will have a characteristic rate of adjustment within which the system will tend to reach a new equilibrium. If the rate is exceeded the system will become unstable and failure will occur either by excessive deformation or by brittle rupture or by both.

Relaxation

Rock is a granular material containing grains of varied yield points, and the grains are often oriented or prestressed so that some tend to yield before others. It is readily apparent from rheological considerations that, if such a medium is loaded, elastic strain energy can be stored in the system and that, if unloaded, some of it will be recovered at once; some will be recovered over a period of time, but a large amount will not be recovered as long as the medium remains a coherent, interreacting, bonded mass.

For example, consider two coil springs of different length, or of different spring constants or of different axial orientations. If the springs are subjected to a given load, then embedded in a polymerizing plastic, a condition of transient equilibrium will be reached between the springs and the bonding plastic after the plastic has set. The springs will tend to push out to their original shape, but will do so in different amounts and perhaps in different directions. One might be in compression, the other in tension and the plastic between in shear. Very high energies can be conserved in such a system.

If a cut is made so that the geometry of the mass is affected, a new state of equilibrium will be required and movement will occur accordingly. Continued cutting would create continued movement.

All rocks have been subjected to changing loads and conditions during formation and since. A rock is, therefore, a product of its own history and all rocks must contain more or less conserved elastic strain energy. Its present condition is a transient one. Therefore when a new free face is created on a rock there will be movement toward a new state of equilibrium. The movement will be in two phases, an immediate elastic rebound largely volumetric, and a time-dependent further relaxation. This will occur first in the actual new surface and will consist of expanding, moving, and rotating grains. As the outer layer relaxes the next layer has less restraint and it will relax somewhat but not as much as the first layer. As relaxation progresses, unless the grains are completely decoupled, a strain gradient is established from the outside into a region where the original strain condition is approached. Relaxation is a reversal of the condition of strain originally imposed on the rock.

This can be useful to engineers because, if a rock sample of known orientation is cut and instrumented so that strain relaxation measurements can be taken continuously on three mutually orthogonal faces, then the relaxation patterns, the relative magnitudes of the relaxation strains, the statistical directions of the principal strains and the rates of relaxation are measurable (5).

Thus it is immediately possible to define the force field in relative magnitudes and directions acting on the rock mass in its present transient state. The strain ellipsoid and the strain trajectories can be plotted. The orientation of the planes along which the rock will break and their relative order of importance can be defined.

Because rocks are granular heterogeneous aeolotropic technical substances with packing patterns and with inherent strain patterns it is necessary to assess their engineering and rheological properties with regard to the location and the time at which the assessment is made.

The so-called engineering "constants" are not constant. Young's modulus is a variable. It is different in different directions in the same rock (6, 7), and it changes with load. As a substance is compressed under restraint it will eventually approach its limiting relative volume. If no further strain can occur E must be a very large number. It must vary under loading both up and down as the load fluctuates. The strain in a rock under a given load is different in the principal directions and so is E. Because of packing patterns there are stages in the loading of a rock when the volume must increase under increased load. This follows from the packing theory for granular media and direct measurements confirm the theory. Similarly Poisson's ratio is as much a function of grain wedging as it is of anything else. If the wedge surfaces are at 45 deg then a μ of 1 could be expected without consideration of plastic flow (3).

HIGHWAY FOUNDATION

A highway is seldom straight for great distances nor does it parallel the strike of the rocks in its foundation except locally on occasion. It passes over various kinds of rock in various states of stress and with various amounts of contained recoverable elastic energy. A rock cut enters a rock and penetrates into some deeper stratum, then back to the surface again during which it crosses well-defined strain gradients. A cut will have different conditions on each wall because of the rock characteristics. For example, the rock may dip out of one wall and into the other. The force field thrust may be out of one wall and into the other.

Roadbed on Horizontal Rock Strata

The simple case where some excavation of loose material is made and a roadbed is built on top of or near the top of a stratum of underlying rock in a more or less horizontal orientation is not always as simple as it seems. The rock and overburden originally form a system in a state of transient equilibrium. The removal of soil acts as a disturbance to the system. The importance of such a disturbance depends on the original state of strain and on the extent of the change in the force field involved.

On removal of restraint the rock surface will tend to relax. In a simple case this may happen with no real deterioration of the rock. There is always movement and it usually varies in direction and amount from place to place. This can result in differential roadbed movement and can cause damage to a rigid road surface. A more usual case is that relaxation and movement occur as noted but in addition the rock surface deteriorates through minute or other cracking, and this may change the planned drainage or some other detail so that the road maintenance problem is accentuated. In some serious instances removal of rock cover will permit the rock to relieve by arching upward. This may happen quickly or over a period of time but in any case it creates maintenance problems. Experience in these conditions has led to the use of rock-bolting, cut-off slots, prestressing and other techniques to permit the necessary movement to take place under control. The use of such aids involves the measurement of the rock characteristics first. This is relatively cheap and quick to do but is seldom done.

Roadbed on Dipping Strata

Any removal of restraint on a rock surface in which the strata dip will cause relaxation. If the strata bedding intersects the rock surface there will be differential relaxation on the different beds because they have different properties. If this is not taken into account in the road design then one section can move relatively to another depending on the orientation of the road with respect to the strata outcrop. This can result in shear or tension cracking in the bed and in the road surface. Sometimes this has been mistaken for bed settlement except where the failure has included an increase in elevation of a point on the road. Under compression a section of the road may fold or shear or both.

Where the road is on the strata so that both bed and strata slope together and restraint is removed, there is set up a down-dip component of strain because of the tendency of the rock to flow in a down-dip direction under its own weight. This is often translated into a slump type of deformation which affects the road. However, a small elastic strain in a down-dip direction can be serious as far as the road surface is concerned. A strain of 100 microstrains can amount to a relative movement of 1.2 inches in a thousand feet. A strain of 1000 microstrains will multiply this figure accordingly. Rock relaxation of amounts in excess of 1000 microstrains are common and a down-dip component usually involves strains of this order or greater in hard rock.

If the road foundation involves cutting a notch in a slope, for example on a hillside, then the whole movement may be greatly increased. Most mountain slopes by their nature are critical or nearly so. Weathered hills are barely subcritical as far as the rocks are concerned. The angle of slope is not a function of the angle of repose of the broken rock but of the state of strain in the whole rock mass and of the material properties of the rock. Again no rational design is possible without some measured data.

TUNNELS

The design of a tunnel should consider both cost and maintenance. It is analogous to the design of a mine opening.

If the axis of the tunnel is on any other orientation than the thrust direction in the rocks then there will be a rotative movement on the tunnel as well as a tendency for the roof to shear past the abutment on the side opposite to the thrust and in the direction of the thrust component.

The expected movement in a tunnel will consist of a series of related movements. The floor will tend to relax and so will the walls and roof. This will take the form of bending longitudinally in a vertical plane, bending laterally in a horizontal plane and rotating laterally in cross section with attendant tension cracks, shears, and compressive spalling. The method of mining, the method of restraint, the design of the lining, if any, the drainage and the orientation of the tunnel will all depend on the rock conditions. These details are well recognized in the design of mines and details of one case have been published (8). Both laboratory and in situ measurements are required.

ROAD SURFACE

Surfacing materials are usually rock fragments bonded or otherwise. The fragments are characteristic of the rock mass from which they originated. All contain some strain energy.

If the aggregate is crushed rock the act of crushing will have relieved the particles of some of their elastic energy because of rebound. If the rock is piled before use the time-dependent strain energy will be restrained and will not completely relieve. Relief will continue in the road surface.

In some rocks such relief serves to crack up the fragments further and so cause deterioration of the aggregate. If the relief is considerable a form of growth concrete results. This is not necessarily chemical although chemical reaction may accompany the relaxation. Strains transferred to the enclosing cement from the expanding aggregate can cause surface spalling and attendant deterioration of the concrete.

Limestones are often sufficiently varied in their inherent strain retention so that one layer will be strongly reactive while an overlying or underlying one will not be. This is particularly so in the Paleozoic limestones in the area near Kingston, Ontario, for example.

Another aspect of roadbed preparation is the fill used and the way in which it is placed. In any rock pile, particularly freshly fragmented rock, there are two reactions of importance. The first is the reaction of relaxation on those surfaces not under load. These surfaces tend to exfoliate and to spall in relaxation. Such reactions may disintegrate the rock in a period of time and slump will occur.

The second reaction is something over which some measure of control can be exerted. At every contact point in a rock fill pile there will be a stress concentration. In a rock already subject to relaxation on the free faces the stress concentration can increase the tendency to spall and to break down the rock pieces. In rocks where relaxation is not so important there will be the problem of deterioration of the rock pieces at the corners of contact. In both cases slump will occur.

The slump can be reduced somewhat if attention is paid to the condition of placing the fill. If fill is placed by tipping off the top, then there is formed a natural stratification of successive layers inclined at the angle of repose. There is also a tendency for large lumps to segregate in rolling. Pressure along the strata will cause more yielding than pressure normal to it. If the fill is placed in horizontal layers and compacted during placement, the slump from both relaxation and stress concentration will be minimized.

MEASUREMENTS

Most of the information required to define the problems mentioned above can be obtained by direct measurements either on laboratory samples or in the field. Usually both are required. The selection of a highway site and its local development will be a compromise based on nontechnical as well as technical information but as much information as is reasonably possible should be available and should be integrated into an optimum design. In the matter of rock mechanics it is required to determine the existing condition of the rocks and then to calculate what will happen to them under the new conditions imposed by the highway construction and operation. A further problem is then to decide on any possible remedial measures to improve the performance of the structure.

Because the techniques of rock mechanics can generally make use of the same surface exposures, drill cores, and underground openings used by the geologist, it is good practice to have the structural geologist and the rock mechanics engineer work in close cooperation.

Laboratory Tests

Much information can be derived from laboratory tests of samples with known orientation. Oriented hand samples from selected points on surface and underground exposures are cut and instrumented on three mutually orthogonal planes. From the relaxation characteristics indicated by the instruments (the writer makes use of photoelastic plastic), it is possible to record: (a) statistical directions of the principal strains on each surface, (b) orientation and distribution of the planes of preferred shear, (c) general pattern of distribution of the elastically recoverable strains on each surface, (d) relative magnitudes of the principal strains on each surface and from specimen to specimen, and (e) relaxation strain-time curves for each face.

From these tests the strain trajectories in any plane in the rock mass can be plotted directly. If the statistical directions of the principal stresses are assumed to be the same as the statistical directions of the principal strains, then the strain trajectories are the stress trajectories. The principal planes are then readily identified and the relative strain energy distributed over them can be estimated. Areas of increasing strain and of decreasing strain are readily identified. Anomalous local conditions will be evident if this occurs.

The directions and orders of magnitude of the planes of preferred shear are important. These are the surfaces along which the rock will break most easily because they represent regions in which shear strain energy is concentrated. Additional load applied to increase the shear strains will cause breakage, but applied to oppose the shear strain, will reduce breakage.

Relative magnitudes and directions of the principal strains from surface to surface and from specimen to specimen will establish the direction of the current thrust in the operative force-field and will disclose strain gradients within the mass. Tunnel design requires this information specifically.

The relaxation strain-time curves give information on the speed of elastic recovery and can be used to estimate the characteristic rate of propagation of strain in the rock. This is important in planning sequences.

Laboratory tests usually include compressive, flexural and vibrational analyses. These do not supply any absolute quantities but do indicate the degree of aeolotropy in the rock and allow comparisons to be made. Such tests must be made with careful attention to the direction of the specimen axes. Relative conditions of failure, degrees of flow, relative no-load moduli and relative coefficients of viscosity can be obtained from these tests. Such relative figures will have validity in the field if account is taken of the effects of mass, restraint, and geometrical configuration.

In Situ Tests

Any laboratory sample by the act of preparing it is changed to some extent. The most important change is the loss of elastic rebound energy. It is generally good policy to measure both the elastic rebound and the time-dependent relaxation on the same sample on some systematic basis. This is easily done by a simple over-coring technique using a diamond drill to overcore a plastic disc previously cemented to the bottom of a hole of smaller diameter. Rebound and the following relaxation are both recorded on the same disc.

In tunnels or road cuts it is desirable to measure the amount of stress multiplication at corners. It is also useful to measure the direction of thrust and the change in rate, magnitude, and direction of the force-field affected by the working. This can be done with rock bolt dynamometers, bore hole gages or surface gages.

A useful series of measurements can be made cheaply and quickly with seismic equipment. Two kinds of information become available. From the travel times of seismic waves the thickness and condition of various rock layers can be estimated. In addition, if used in conjunction with laboratory knowledge of the directions of principal strain and the planes of preferred shear, the travel time information can be used to design blasting patterns with high productivity and low energy ratios. In several instances within the experience of the writer a very simple in situ measurement has resulted in a radical improvement in the excavation costs and character.

Seismic methods are sometimes used to determine moduli but it should be remembered that from rheological considerations a seismic test is usually a short-time test and that the rock affected by it will behave more elastically if given no time to flow, whereas in actual operation flow is usual. Therefore the moduli found will be too high except for calculating other dynamic effects. Such a modulus must be corrected for changes of load and must be used only with regard to the orientation of the direction in which it was measured.

A new procedure in use and under test for the past two years involves a combination of a bore hole gage and seismic techniques. The writer has used a high compliance photoelastic plastic plug which is glued into a bore hole at any depth desired. The plug has its own light source embedded in it and the biaxial strain field normal to the plug axis is represented by a fringe pattern resulting from the retardation of the light at the source by the strained plastic. Such a pattern gives on inspection the direction of principal strains and their relative magnitudes. If viewed from time to time any changes can be recorded.

Viewing of the plug can be direct with a telescope at distances of up to 100 ft, or as long as bends in the hole do not cut off the view. At greater depths a small bore-hole camera will record the picture and its orientation in color. The strain can be monitored at any time with a photocell and a recorder.

An additional advantage of the plastic bore hole plug is that it will also register any seismic disturbance. A photocell scanner will plot the frequency and amplitude of a series of vibrations set up by any mechanism. If a shot point is used then the time of arrival as well as the wave form characteristics and damping can be directly determined. The plug remains at the same spot and it will continue to record any time-dependent changes. If at a later date another blast is detonated at the same point as before, any change in rock characteristics since the last record will be indicated by a different reading characteristic in the gage. Some gages have now been in continuous use for three years but improved models have been used for the last two years.

It is perhaps noteworthy that the same gage and also a low-compliance high-modulus photoelastic glass plug have been used successfully to monitor the strains in concrete structures by highway engineers in England.

In situ measurements are warranted prior to construction because of the low-cost, highly reliable design data that can be obtained by these techniques. Similar measuring devices are also useful during and after construction to test the validity of the calculations and of the predictions based on earlier measurements. If everything is seen to move or strain at the predicted rates and in the predicted amounts and directions, then the structure is under control. If any unexpected condition occurs then the structure is out of control. However, the engineer will at once be aware of the problem and remedial measures may be taken well ahead of catastrophe. The effects of the remedy can be seen on the same instrument.

CONCLUSIONS

It has been the intention to point out some of the problems in highway design caused by rock conditions and to indicate the nature of simple tests that can be used to provide data to enable a more rational design to be developed where rock is concerned.

Recent applications of the instrumentation used by the writer have provided good results in England and in Canada on highway structures, both concrete and rock. The cost is low and the procedure can be operational at the site. There seems to be no valid reason for not making use of present knowledge in rock mechanics. In this respect there are now many mines in the USA and elsewhere with the design based on the rock mechanics theories mentioned here. The theories are no longer academic hypotheses but are working theories. The instruments are proven and operational. It is time to apply this knowledge on the design of any functional structure in or on rock.

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Stability of High Road Bank Slopes in Rock—Some Design Concepts and Tools

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The instruments and data analysis procedures used by the Bureau of Mines in a continuing study of predicting the steepness of the angle and height at which a rock slope will safely stand are described and discussed.

The instruments and procedures represent the beginning of a scientific effort to quantify experimentally the engineering and general significance of the attitude of rock joints, bedding planes, and faults; the character of gravity and tectonic induced stresses in rock slopes with and without berms; slope geometry; ground water; and overblasting as they affect the stability of a rock slope. Also discussed is the use of the pre-splitting technique of blasting to form a smooth and tight slope face relatively free of rock falls.

The reporting of these mining research studies may provide highway engineers with information regarding some new rock slope design tools which they may be able to use. However, this segment of rock excavation science is in its infancy, and much costly and time-consuming work must be done before the various rock slope stability factors can be assigned numbers. Ultimately, technically sound and rational rock slope design criteria and procedures will be developed.

•DEEP CUTS in rock requiring the excavation of several hundred thousand or more tons of rock are becoming more commonplace as networks of wide multilane and limited-access Interstate highways spread across the nation. For these deep cuts the highway engineer is faced with the problem of estimating the slope angles for the sides of the cuts. If the selected angle is too steep, the uninterrupted usage of the highway and public safety is endangered by excessive and persistent rock falls or road-obliterating slope failures and rock slides. If the planned slope is too flat, the projected cost of excavation may lead the engineer to resort to the unnecessary expense of chosing a longer route or driving a tunnel.

The mining engineer is confronted with a similar problem in open-pit mines. In these mines the steepness of the pit slope is a major economic consideration because the angle of the pit slope predetermines the quantities and the cost of removing waste material needed to safely recover ore.

Minimizing the amount of rock to be excavated is an objective common to both the mining and highway engineer in designing rock slopes. Solution of the problem requires that a quantified knowledge of the material properties, stress distribution, and effects of environmental factors be available. Unfortunately, the technology of quantifying the en masse properties of rock materials, the assessment of the effects of environmental factors, and the stresses to which rock slopes are subjected in open-pit mines and road cuts are, at this time, inadequately developed to positively establish the slope design criteria needed. Consequently, rock slope design practices have been and still are largely based on experience and "cut and try" methods, with slopes frequently made too steep or too flat. As such, large expenditures are often necessary to correct slope pitch after a failure occurs or for the unnecessary removal of more rock than required.

Recognizing the importance for developing a better engineering understanding of the factors governing rock slope stability, the Bureau of Mines and Kennecott Copper Corporation independently, about 7 years ago, initiated investigations in the application of the principles of soil and rock mechanics in the design and control of rock slopes. In these initial studies, both groups assumed that the principles of soil mechanics, with possibly some slight modification, might be applied. However, the studies indicated that the application of soil mechanics was limited, in open-pit mines, to slopes composed of soil or to those parts of the active slide areas in which the broken and moving rock mass possesses physical properties similar to soil. Perhaps most important was the indication that combinations of rock structures were primary factors in the stability of a rock slope, with water seepage into and through the structures acting as an agent in promoting failures.

In 1960 the Kennecott Copper Corporation planned and started an extensive applied study of the stability of pit slopes using the Kimbley pit near Ely, Nevada, as a full-scale experimental model. Because the program paralleled a general pit slope research study of the Bureau of Mines, Kennecott invited the Bureau to participate in the applied work as a joint Kennecott-Bureau effort. Some of the results of work completed and in progress have been published (5, 6, 13, 14, 22), or are being prepared for publication.

The following sections of this paper describe and discuss some of the open-pit mine rock slope design concepts and study procedures derived in the joint Kennecott-Bureau and related Bureau rock slope stability studies with the emphasis on those ideas that highway engineers may find useful in estimating the steepest slope at which road cuts may be expected to stand.

PRINCIPAL ROCK SLOPE DESIGN FACTORS

Because we must consider a pit or road cut rock slope as an engineered structure, its design is primarily a problem in applied mechanics in which the factors to be considered, whether they be expressed by numbers established by measurements or estimations based on empirical knowledge, may be generally categorized as (a) stress geometry, (b) rock structure, (c) environmental conditions, and (d) excavational procedures in sculpting the slope.

Stress Geometry

In theory and in practice the structural stability of any excavated opening in rock is partially dependent on the stress field, i.e., the state of the stress in the rock before excavation, the stress distribution in the rock created by the excavation, and the in situ structural strength of the rock.

All rock is in a state of stress before openings are excavated. The stresses may range from near zero to many hundred pounds per square inch, depending on the depth and configuration of the opening and on the magnitude of tectonic forces to which the rocks may be subjected.

The opening created by the excavation of a pit or road cut affects the stresses in the rock near the pit or cut. The stress distribution created by the opening is commonly referred to as a stress concentration, which is defined as the ratio of the stress at a point to the stress located outside the zone influenced by the opening.

The problem of evaluating the slope stability effects of stress concentrations created in the walls of an open pit or road cut become quite complex when the opening is large and irregular in shape, the elastic properties of different rocks in the slope vary widely, and planar discontinuities occur in the rocks in or near the slope walls.

Because of these complexities there is no satisfactory mathematical or physical model that can be used to predict these stresses or stress concentrations or their quan-

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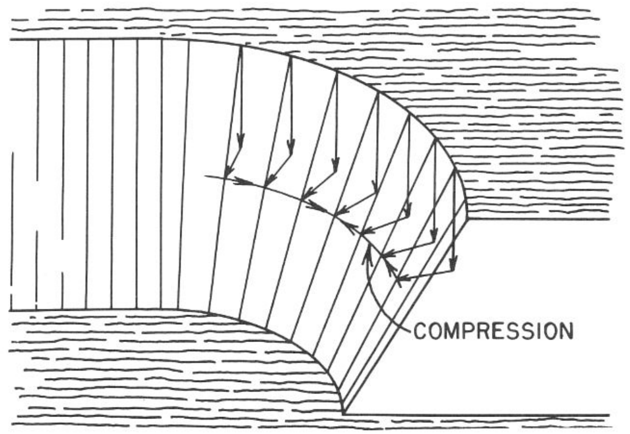


Figure 1. Strengthening effect in a concave-shaped wall of a slope as the result of horizontal tangential stress created by the lateral resultant of the wall burden load.

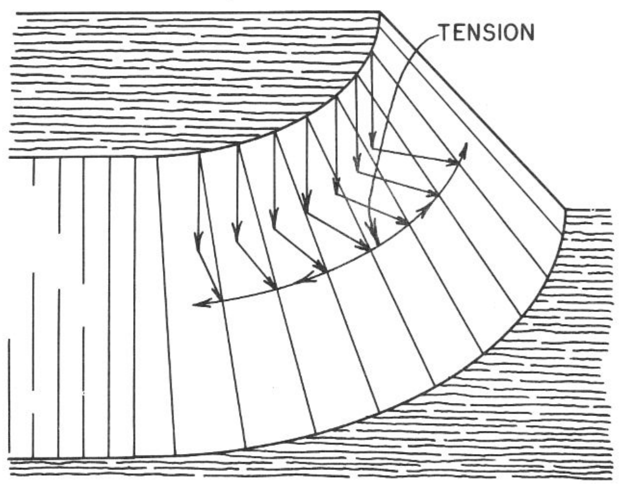


Figure 2. Weakening effect in a convex-shaped wall of a slope as the result of the horizontal tensional stress created by the lateral resultant of the wall burden load.

titative effect on the stability of a rock slope. Therefore, the concepts of the distribution of stress and the manner in which stresses affect the stability of rock slopes presented in this paper are based on (a) the qualitative stress distribution and concentration information derived from a laboratory study of body-loaded, two-dimensional, photoelastic model slopes; (b) some computed stresses as derived from measurements made in stress-relieved boreholes drilled horizontally at several sites into the walls of an open pit; and (c) some simplified assumptions as applied to the consideration of stress in, and in the vicinity of, an excavated surface opening as a pit or road cut slope. Briefly stated, the general stress assumptions are as follows:

1. Vertical stress in rock which increases with depth from the surface is generally due to the cumulative body weight of the rock. It is usually about equal to the overburden weight density times the subsurface depth.

2. Horizontal stresses are developed by lateral constraint of rock loaded by the overlying material, by tectonic forces, or by both.

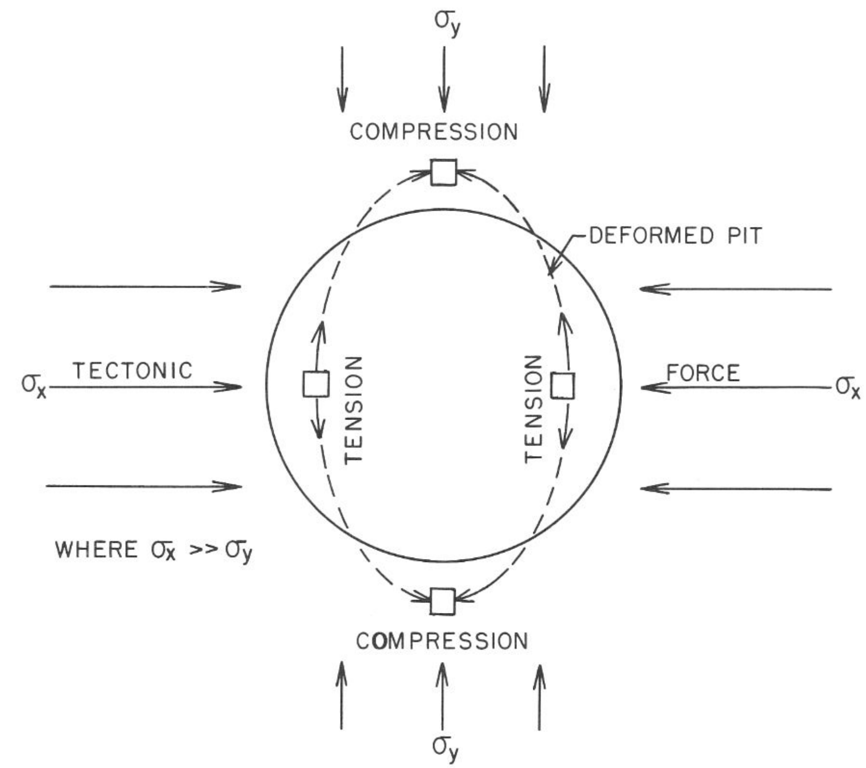


Figure 3. Exaggerated depiction of deformation of a circular pit resulting from an excessively high horizontal tectonic force.

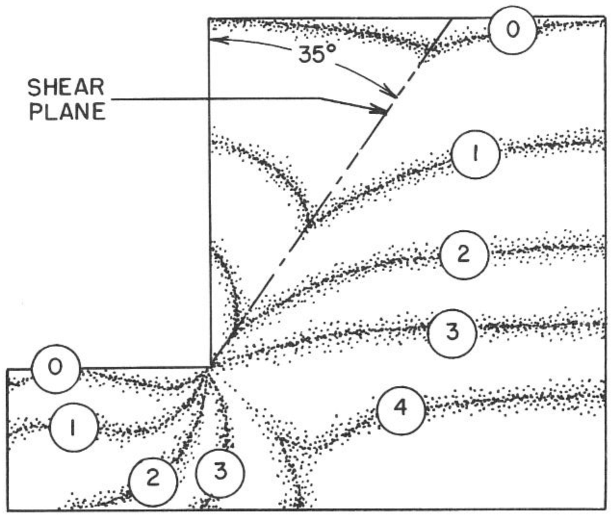


Figure 4. Levels of strain and shear plane developed in a photoelastic model of a vertical slope.

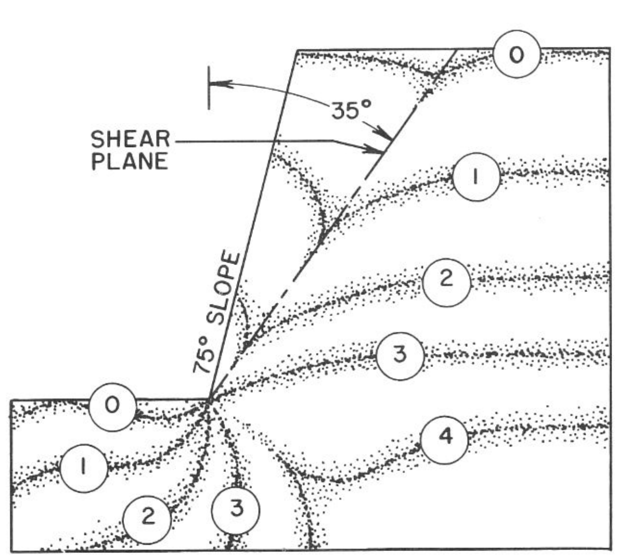


Figure 5. Levels of strain and shear plane developed in a photoelastic model of a 75-deg slope.

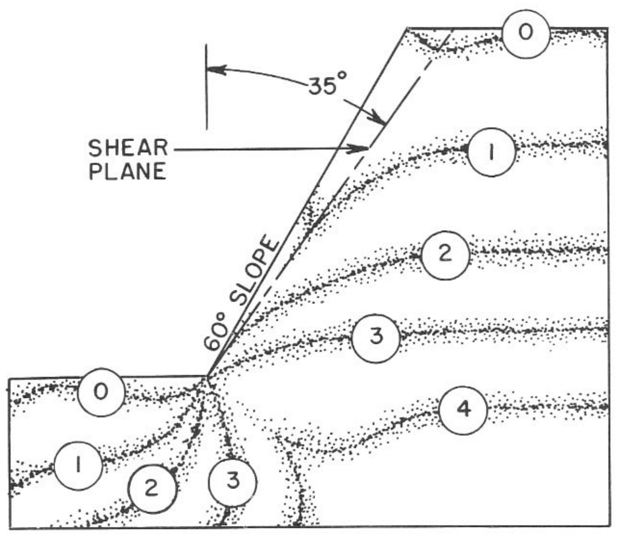


Figure 6. Levels of strain and shear plane developed in a photoelastic model of a 60-deg slope.

3. Horizontal stress should produce stresses in the slope which act parallel to the horizontal plane and tangentially to the rock slope surface. These tangential stress concentrations in a slope face may favorably or adversely affect the stability of the slope, depending on whether these stress concentrations are zero, tensile, or compressive.

4. In slopes that are concave in plan, the horizontal tangential stresses tend to be compressive (Fig. 1). In slopes that are convex in plan, the horizontal tangential stresses tend to be tensile (Fig. 2).

5. Rock is relatively weak in tension, and tensile stress concentrations in a slope would promote instability; horizontal tangential compressive stresses, within reason, should assist in slope stability by increasing friction and restraint along fractures.

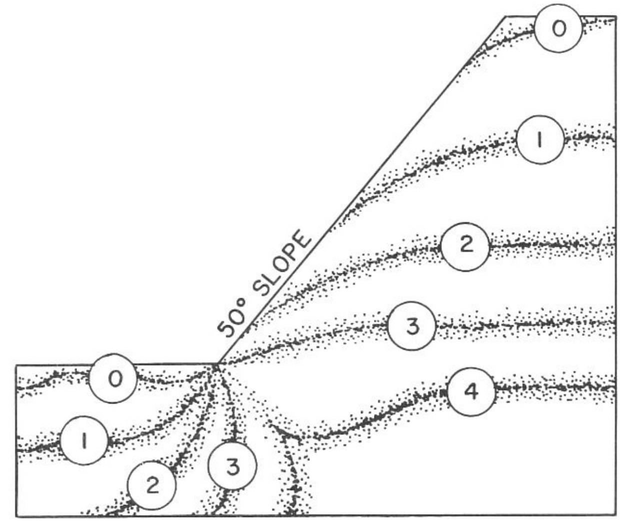


Figure 7. Levels of strain developed in a photoelastic model of a 50-deg slope.

6. Stress concentrations around a pit or in a concave road cut slope can be tensile even though the two-dimensional stresses in the horizontal plane are compressive. This can be demonstrated by assuming a circular pit in which the rock (in mass) reacts as an ideal elastic medium and the stress concentrations created by the pit are equivalent to those created in a circular opening in a semi-infinite thin plate in a biaxial stress field where a compressive stress S is five times greater than a compressive stress T. Under these hypothetical conditions, the tangential stress concentration at two points on the edge of the opening is tensile and equal to -2T (1, p. 80). An analogous condition in rock slopes could occur should a horizontal tectonic force of sufficient magnitude be superimposed on one of the lateral and horizontal components of the vertical overburden load. A stylized and exaggerated illustration of this condition is shown in Figure 3.

7. Where any relatively incompetent rock or weakness created by joints, faults, or other planar discontinuities in a rock slope is subjected to tensile stresses, failures would be more probable than if these same areas were in confinement created by compressive stresses.

These are the assumptions and the hypotheses on which a qualitative evaluation can be made of the effects of stress that, in part, govern the stability of rock slope.

As previously stated, a preliminary estimate of the distribution and the points at which the gravity-induced stresses tend to be concentrated in a slope was obtained through a study of photoelastic models. Figure 4 is a simplified drawing of a photoelastic model representing a vertical slice from a hypothetical vertical-walled pit in which the gravity-induced unit strains are shown as stippled bands. Each of the bands represents a zone in which the strain is of a specific order of magnitude with the order of magnitude indicated by the circled numbers. The convergence of bands at points indicates the areas in which stress is concentrated. Thus it can be seen that stress is concentrated in the toe of the slope and that the stress in the floor near the slope toe is a tensional stress. The line through the inflection points of the top three bands denotes the plane of the angle of shear developed in the model. It can be seen that this plane starts at the toe of the slope wall and trends upward at an angle of 35 deg to the vertical. In Figures 5 and 6 this line is also at an angle of 35 deg, even though the slope faces are now at angles of 75 and 60 deg, respectively. As seen in Figure 7, no such line of shear develops in a 50-deg slope.

Other model experiments also have shown that the magnitude of the stresses increases as the height of the pit wall is increased, but the angle of this line of shear remains at

TABLE 1

STRESS DETERMINED FROM MEASUREMENTS NEAR THE EDGE OF THE KIMBLEY PIT

Site	Depth from Slope (ft)	Maximum Stress (psi)	Minimum Stress (psi)	Direction[a] (deg)
1	12 to 21	113	103	96
1	69 to 71	131	51	176
2	12.5 to 21	150	90	178
3	66 to 90	520	415	2
3	100 to 105	496	300	4

[a]Angle between the right-hand horizontal and the maximum stress.

35 deg. It should be pointed out that the 35-deg angle developed is the angle representative of the homogeneous, isotropic material from which these models were constructed. It has not been established what this shear angle might be in a rock slope.

However, on the basis of the model-derived information, it is evident that, in a particular type of rock, the magnitude of the shear stress would increase proportionally with the increase in slope height but the angle of shear would remain constant regardless of the height of the slope. The model studies also show that as the height of a pit wall is increased, the increase in the stress is greater than the increase which would result from steepening the angle of the slope.

Figures 1 and 2 show diagramatically how the gravitational loading of the rock in slope walls and the resultant lateral stresses are transformed into a tangential stress at and near the slope surface.

Where the slopes form a concave surface, the tangential stresses act as a "hoop stress" and the blocks of rock in the pit wall tend to be squeezed together (Fig. 1). This creates an arch-like effect, thereby increasing the stability of the slopes. The converse is true in the convex-shaped slope face (Fig. 2).

A series of measurements made by the Bureau of Mines in the wall of an open-pit mine revealed that tangential stress concentrations exist in enclosed, concave, pit configurations. The results of these measurements and calculations are summarized in Table 1.

Because the boreholes were horizontal and normal to the pit slope, these determinations provide estimates of the stress in a plane parallel to the slope. Except for the measurements between 12 and 21 feet at site 1, the maximum of the two orthogonal stresses is almost horizontal. Further, because sites 1, 2, and 3 are at deeper elevations from the original ground level, there is evidence that the tangential stresses increase as the distance from the ground level increases. Of note is the fact that the field stress determinations indicate that the expected horizontal stress at site 3 should be about 300 to 350 psi. The horizontal stresses determined at site 3 are about 500 psi; therefore the stress concentration (stress at site 3 divided by the horizontal stress) is about 1.5.

The above measurements were made in holes drilled horizontally at three sites into the walls of an open pit to depths ranging from 21 to 105 feet in length using the borehole deformation gage and stress-relief overcoring method developed by the Bureau. The instruments and the measurement and stress computational procedures used have been described and explained in published articles and reports (2, 8, 9, 10, 21, 22). Soon to be published (11) is an alternate method that may be used to compute stresses from strain measurements made by the overcoring method.

Information derived from the photoelastic models also shows that no overall reduction in slope stress is achieved by berming a rock slope face. As shown in Figure 8 the level of stress at the roadway toe of the slope is no greater than the stress would be had the slope face been continued to its full height in a straight plane unbroken by a berm.

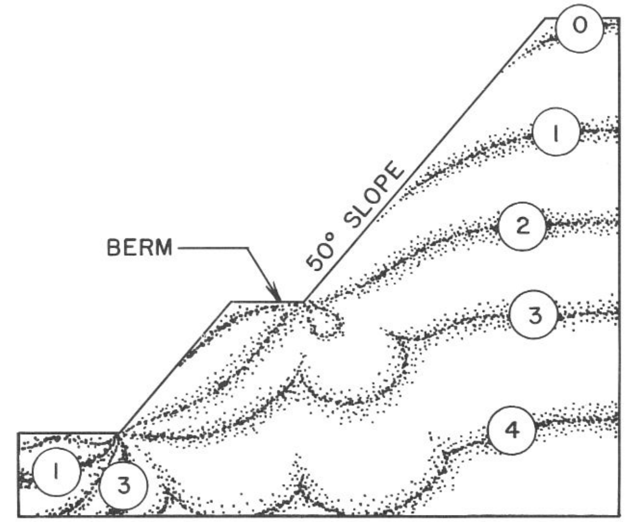


Figure 8. Levels of strain developed in a photoelastic model of a bermed, 50-deg slope.

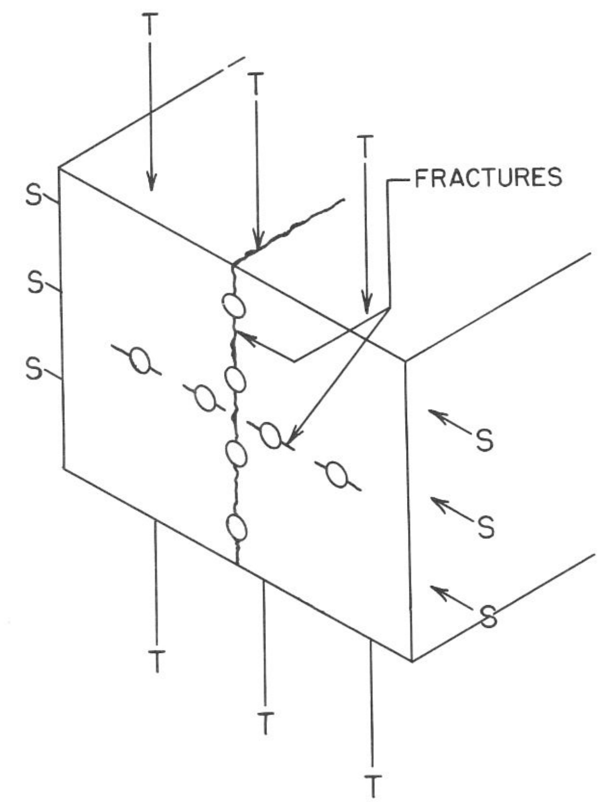


Figure 9. The effect of a nonuniform stress field on the establishment of fractures between boreholes by pre-split or smooth-wall blasting.

The extra rock that must be removed to include one or more berms appears to be a needless expense as far as the stability of a slope is concerned. Thus the only purpose of a berm is to catch fall rock. Generally berms to catch fall rock can be largely eliminated by using the presplitting or smooth-wall blasting techniques to form the slope face. Numerous articles and reports covering the theory and application of presplitting have been published (12, 15, 17, 18, 20), and the forming of slope faces in rock by the method is rapidly being accepted as a standard procedure.

However, the adverse role that horizontal tectonic forces may play in the achievement of a smooth face by presplitting has not been equally well publicized. Referring to the previously stated simplified stress concentration assumption, number 6, we see that a nonuniform stress field may result where a tectonic force is superimposed on the normal lateral component of the vertical stress. The effect of such a nonuniform stress field on the establishment of a presplit fracture between adjacent boreholes is depicted diagrammatically in Figure 9. The length of arrows labeled S and T denotes the relative magnitude of the stresses as applied to the block representing a segment of rock in which holes have been drilled and in which presplitting charges have been detonated. Fractures established by the blast parallel the S (the trajectory of the greater stress level) direction with fractures established between holes in the S direction and not between holes in the T direction. To establish fractures between holes in the T direction the holes must be loaded with much heavier charges than the holes in the S direction. Thus, to be able to specify hole spacing and charges to successfully presplit at any particular site, the planning engineer must know whether the horizontal stresses at the site are uniform or nonuniform.

The Bureau has found relatively large horizontal stresses in igneous, metamorphic and sedimentary rock areas. In some instances these stresses are not uniformly distributed in all directions (8, 3).

The level and trajectory of the tectonic stresses in a particular area may be established by the borehole stress relief method developed by the Bureau. Generally the information can be obtained by stress relieving one hole drilled vertically into the rocks exposed at each suspected site to such depth as required to penetrate below the zone affected by sheeting or surface fracturing. Where rock is solid and unaffected by sheeting or surface fracturing, the hole need not be more than 3 or 4 ft deep.

Rock Structure

In the initial phases of the slope stability investigation conducted by the Bureau, a detailed study of slope failures in open-pit mines revealed that the incipient plane of failure and the plane along which movement of the mass of rock ultimately took place was almost always a planar or structural discontinuity in the rock. This observation led to a study of natural rock slopes; in all the slopes studied the rock structural features were found to be the primary factor governing the stability of the rock slopes. This conclusion was also reached by Terzaghi (19) in his assessment of factors governing the stability of slopes in unweathered rocks. Therefore, the Bureau has taken as a basic working hypothesis the idea that rock structures, as joints, bedding planes, schistosity, faults, etc., are measurable features which can be utilized as a means of predicting the steepness of the angle at which a slope in rock will stand.

Thus the knowledge of the orientation and character of these planar rock structures is necessary in order to establish the steepest angle at which a rock slope will stand. Studies have shown that the critical height and steepness of slopes in relatively unweathered rock may be dependent more on the orientation of the planar defects than on the strength of the rock itself. The engineering concept that all rocks are similar probably stems from the idea that there is relatively little variation in the laboratory-determined compressive strength, and to some extent the shear strength, of most rocks encountered in rock excavational operations. This concept relies too heavily on strength factors determined by the laboratory testing of small samples. These laboratory results can differ greatly from the en masse characteristics that rocks exhibit in place. Hence, the laboratory-determined values of rock strengths must be used with some reservation in computing the strength and stability of slopes because of observed effects of joint planes and other planes of weakness in rocks.

Figure 10. Blocky nature of the slope bank of a road cut in a jointed rock.

Joints subdivide rock into individual blocks which almost fit each other, and the cohesive bond across an uncemented joint is assumed to be zero. Joints may be continuous or discontinous and generally form a three-dimensional network that transforms the rock into a cohesionless aggregate of cuboid blocks somewhat comparable to closely fitted blocks in a dry masonry wall. The blocky nature of the rock in an excavated slope is shown in Figure 10. Unlike a dry wall, the joints in rock are commonly filled with decomposition materials that are prone to turn into a grease-like substance when wet. Thus any set, or intersection of two or more sets, of more or less continuous joint planes is a potential surface along which sliding can occur. Hence, the orientation of persistent sets of joints with respect to an actual slope must be seriously considered.

These same statements apply even more to fault planes, regardless of how great or small the displacements along them appear to be. Joint sets, bedding planes, and fault planes that dip into the slope face will generally have a minor effect on reducing the stability of rock slopes. Conversely, those sets of joints, bedding planes, faults, and other planar discontinuties with dips that are oriented toward the open face of a slope must be carefully considered. The detrimental effect on the stability of a slope increases as the strike of the joints becomes more nearly parallel to the slope wall. The fact that rock is almost always jointed in a preferential direction is all the more reason why it is more than probable that any slope will have one or more sets of joints adversely situated in relation to orientation of slope face.

Therefore, it is essential to collect as much information concerning the dip, strike, location, and character of all bedding, and joint and fault planes, especially the faults, as is physically possible. In addition, the location of breccias and decomposed, weak, permeable materials should be given special attention.

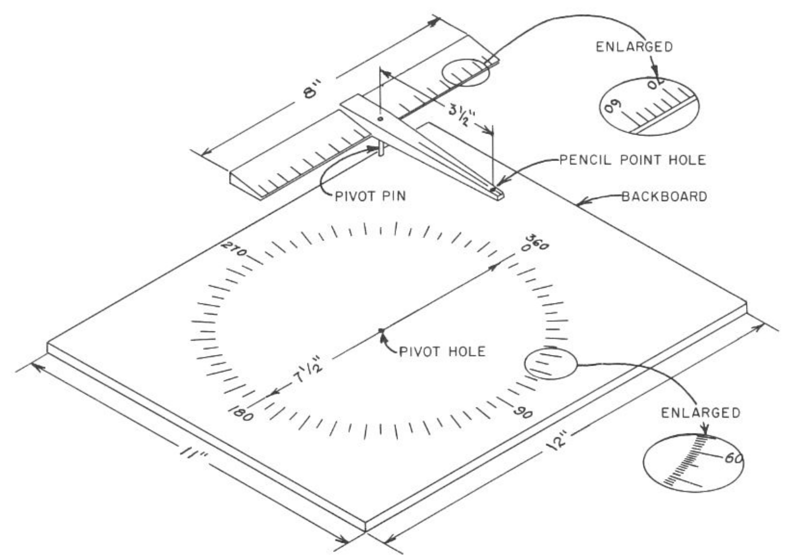


Figure 11. Exploded isometric drawing showing details of the rock structure dip point plotting device developed by Bureau of Mines.

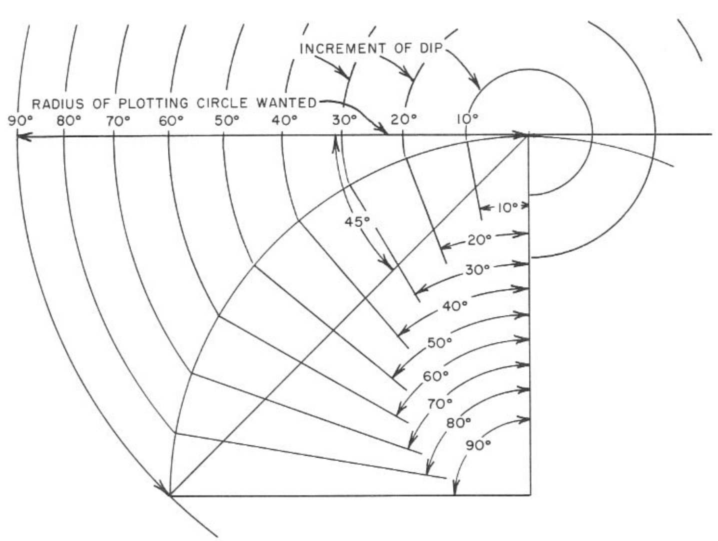


Figure 12. Development of dip angle circle increments of a Lambert's equal-area polar projection for any selected size, lower hemisphere plotting net.

The major part of field data on the rock structures may be collected by conventional geologic mapping methods augmented by data obtained through a study of aerial photographs using photogeologic techniques applied to the mapping of joints as reported by Hough (4). Also, where suitable drilled holes are available additional joint information may be obtained with a borehole camera. The borehole camera and its use has been described by Rausch (13) and Hubbard and Rausch (5).

The dip and strike information may be represented in lower hemisphere equal-area (polar) contoured plots as described by Robinson and Lee (16), or recorded on computer

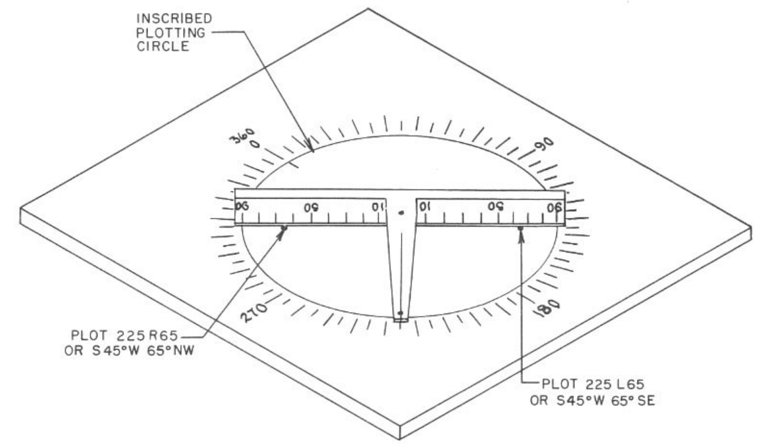


Figure 13. Plotting the planar structure measurements 225 R65 and 225 L65 on a polar net using the Bureau plotting aid.

punch cards which can be machine-sorted by strike and dip and histograms or on contoured equal-area plots made as reported by Hubbard and Rausch (13).

These plotted or contoured diagrams then serve to show the engineer the spatial orientation of the planar discontinuities he must consider as potential sliding planes in designing a rock slope.

A good deal of the tedious work and errors in plotting the dip and strike point information on polar (lower hemisphere) equal-area diagrams may be eliminated by adopting a system of field data recording and a plotting aid developed by the Bureau.

A compass having a full circle 360-deg azimuth is used instead of the quadrant type of circle division normally used by geologists. The dip point plotting device developed by the Bureau is shown in Figure 11. Inscribed on the backboard is a 360-deg circle. In the center of the circle a pivot hole is provided into which the pivot pin on the T-shaped spaced piece fits. On the diametral or longest part of the T-shaped piece there is inscribed from the center to the right and to the left from 0 to 90 the dip angle units corresponding to the polar circles on a Lambert's equal-area polar net. (To provide support for the pivot pin the central 0 to 5 deg is omitted on the scale as structures having a dip of 5 deg or less can, except in very rare circumstances, be ignored as having little or no significance on the stability of a slope.) The method that may be used to derive the spacing of the polar circles to fit any selected size plotting circle is shown in Figure 12.

The method of field notation using the 360-deg azimuth compass is as follows: Strike is always read from the north end of the compass needle as the direction in which the compass is pointed and the operator is facing, and dip as being either to the right or to the left of the direction the operator is facing along the strike plane. A comparison of the manner in which the strike and dip data are recorded in the field notebook using the quadrant and Bureau system is shown in Table 2.

Using the Bureau plotting aid and notation system, the office procedure in plotting the field data 225 R65, shown in Table 2, is as follows:

TABLE 2

COMPARISON OF QUADRANT NOTEBOOK STRIKE AND DIP NOTATION WITH 360-DEG METHOD USED BY BUREAU OF MINES

Bureau System		Quadrant System	
Strike	Dip	Strike	Dip
225	R65	S45°W	65° NW
110	R70	S70° E	70° SW
225	L65	S45°W	65° SE
110	L70	S70° E	70° NE

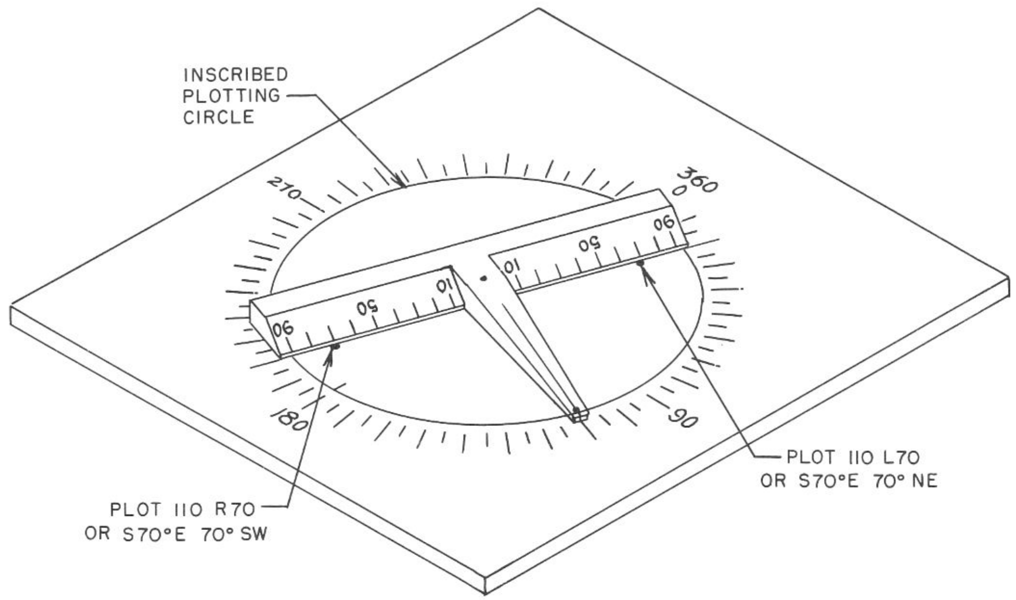


Figure 14. Plotting the planar structure measurements 225 R70 and 110 L70 on a polar net using the Bureau plotting aid.

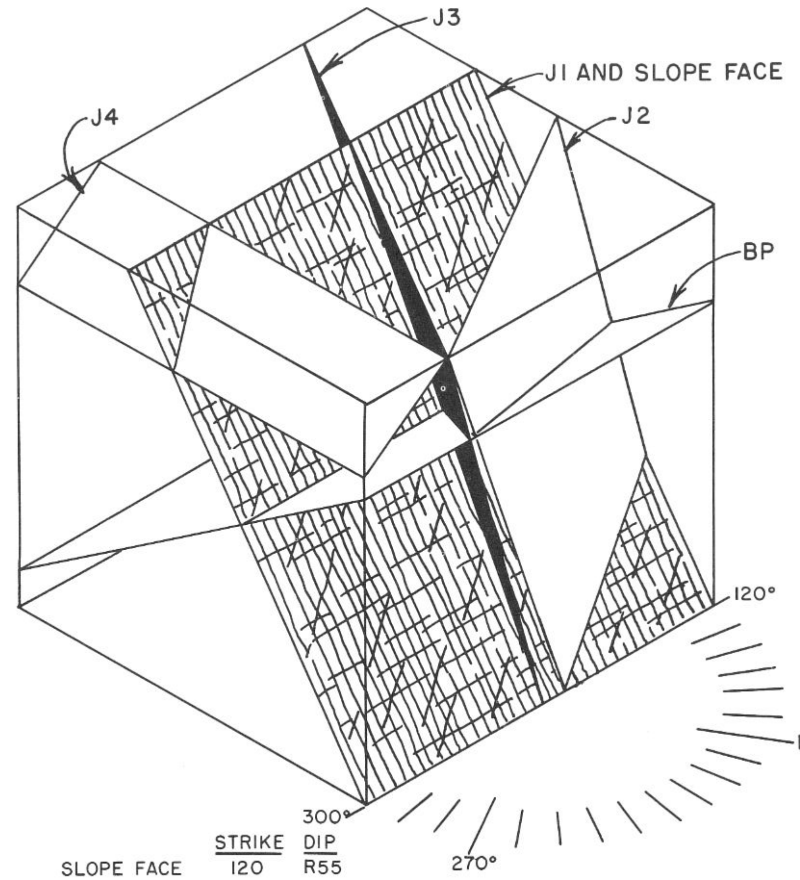
TABLE 3

CONVERTING DIP READINGS FROM LEFT TO RIGHT OR CLOCKWISE DIP DIRECTION

Bureau Field Notation System		Notation System Modified for Computer	
Strike	Dip	Strike	Dip
225	R65	225	R65
110	R70	110	R70
225	L65	45	R65
110	L70	290	R70

1. Fasten piece of tracing paper to board;
2. Mark North or zero point;
3. Insert pivot pin on T-shaped piece into center pivot hole;
4. Insert pencil point in hole in the pointer and inscribe outer 90-deg dip circle;
5. Turn pointer to 225 deg; and
6. Plot dip point at 65-deg mark on the scale on right-hand side of pointer.

Additional points are plotted in the same manner with dip points plotted either to right or left of the pointer in accordance with field book notation of R or L. (See Fig. 13 showing the plotting of 225 R65 and 225 L65 and Fig. 14 showing the plotting of 110 R70 and 110 L70.) With this plotting aid and notation sysyem, a nonprofessional worker can be trained in a few minutes to plot accurately rock structural data on a Lambert's lower hemispherical equal-area net. Using such a dip pole plot, a contour diagram that permits the joint pattern and joint attitudes to be easily visualized (16) may be manually prepared by following a unit area counting procedure that is commonly used by geologists in analyzing petrofabrics data. If, however, the analysis of joint plane data is to be done by a computer, the Bureau system of strike and dip notation should be modified so that all dips are recorded as being in a direction clockwise from the strike. This simplifies programming and can be easily accomplished in the field by taking the reading at the south end of the compass needle as the strike direction for what would be considered a leftward dipping plane in the Bureau field notation system. The difference in notation is shown in Table 3.



	STRIKE	DIP
SLOPE FACE	120	R55
J1	120	R55
J2	90	R84
J3	60	R70
J4	30	L45
BEDDING PLANE	120	L35
SLOPE HEIGHT	110 FEET	

Figure 15. Block diagram showing the spatial orientation of joint and bedding planes relative to the angle of slope in the road cut illustrated in Figure 10.

To read the manually contoured plot, tracing paper is placed on the plotting board with the zero point on the paper aligned with the zero on the board; the T bar is then used to determine the strike and dips of the dominant joint sets.

Figure 15 is a block diagram of the mapped joint and bedding planes in the Hungry Horse rock slope shown in Figure 10. It diagrammatically shows that joint plane (J1) is the control plane in this specific road cut.

Establishing the steepness of rock cut slopes on the basis of orientation of the planar structures relative to the slope face will permit the highway engineer to design stable slopes with a reasonable degree of confidence. An indeterminate safety factor is "built in" because cohesion along the planar structures is considered to be zero.

A more refined system of establishing rock slope angles will eventually be developed when, through continuation of slope stability, research engineers will succeed in developing the necessary mathematical or physical models. By these means the effects of stress distribution and other factors affecting slope stability in rocks may be quantitatively equated.

Environmental Conditions

In many cases, the toe of a slope is located below the water table, and the influence of hydrostatic pressure is not clearly understood nor properly estimated. However, slope failures due to rapid and wide fluctuations in the elevation and configuration of the water table are quite common.

In relatively unweathered, jointed, and fissured rocks, the volume of the open joints in which the water collects and along which the water travels through the rock is very small as compared with the volume of rock located between the joints. Hence, the rate at which the water moves through a rock mass is generally much slower than the rate of movement through pervious, unconsolidated sediments. Therefore, the vertical fluctuations of the water table in a rock mass are much more pronounced than the variations of the level of a water table in pervious sediments. Thus, the vertical distance between the lowest and highest position of the water table in a rock slope may vary by several tens of feet or more as compared with the few feet of variation noted in a pervious sediment. The character of the sometimes quite rapid change of the water table in a rock slope is shown by comparing line A-F with line A-G in Figure 16. In reality, the water table is not as well defined in rock as shown by the dry season line A-F, and the wet season line A-E, because the spacing of joints may vary from place to place and water may rise to different horizons in adjacent observation wells.

The following discussion points out some of the reasons why water in pit or roadcut slopes can present problems that are sometimes difficult to overcome.

During rainy seasons or spring snow melts, part of the water is temporarily retained in the weathered top layer, and the remainder flows as runoff sometimes toward and over the edge of the slope. Before it reaches the lip, the runoff crosses the upper surface (C-B, Fig. 16) of the wedge-shaped body A-B-C. Because of the shearing stresses commonly prevailing in this segment of the pit slope, the joints may be wider and more numerous than those in the rock located farther from the face of the slope. Hence, the quantity of water which can enter the joint system in this wedge, per unit area of its top surface, is much greater than the amount that enters the joint system elsewhere. Consequently, the water table in the wedge may rise temporarily to a position like that shown by the line A-G on Figure 16. This added height of water thereby exerts onto the walls of the pit an added pressure equal to the hydraulic head created by the increase in the height of the water table in the wedge-shaped segment A-B-C. This hydraulic pressure corresponds to the pore-water pressure in soil mechanics and is here called

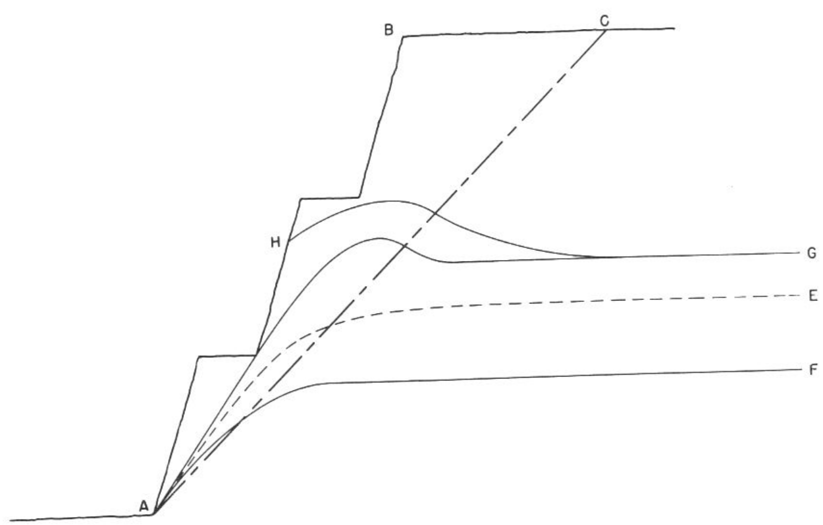


Figure 16. Climatic-induced changes in configuration water table in the slope wall of a road cut or open-pit mine.

joint-water pressure. If the rock near the toe of the slope is already stressed to a near-failure point, the added joint-water pressure may be sufficient to cause the slope to fail. Failure will start at or near the toe, and, as a result of the initial failure, the rock located above the seat of failure will be deprived of support and will fall because of its own weight.

Under certain climatic conditions, an even more destructive joint-water pressure can develop behind slopes. During spring thaws, the night temperature is usually great enough to freeze water. Ice forming in the joints on the face of the slopes effectively seals the joints, thus damming up the thaw water, as shown by line H-G in Figure 16. This increased head of water, although generally temporary, may cause the slope to fail.

The effects of water as a lubricant along gouge-filled fault planes should also be considered where the dip of the plane is toward the open face of a slope.

The effectiveness of surface and underground drainage in stopping or reducing slope movement has been demonstrated many times. In some cases, however, the expenditure of excessive amounts of manpower and money to obtain effective drainage may not be warranted.

Excavation Procedures in Sculpting the Slope

Among the procedures used in the excavation and operation of a slope, the practice of overbreaking at the toes to reduce shoveling difficulties should be considered as a possible cause of slope weakening. The unit strain lines shown on Figures 5, 6, 7, and 8 as dotted bands show the area at the toe of a slope or bench to be more highly stressed than other contiguous parts of the slope. This highly stressed toe is the area where failure of a slope is most likely to start. The shattering of the rock in the toe of the slope by overbreaking unnecessarily weakens the slope at a point where a concentration of stress may have already reached a critical level. This shattering is an aspect of slope stability research that has received too little attention. However, the practice of shaping slopes by the presplitting or smooth wall blasting technique can greatly reduce the weakening of the toe of the slope by vertical shot overbreaking.

CONCLUSION

Through continued studies and research, engineers will eventually develop the necessary mathematical or physical models that will permit them to quantify the effects of stress distribution and other factors affecting the steepness at which rock slopes will safely stand. In the meantime and until such models have been developed, the engineer must continue to design rock slopes and the method of basing the steepness of rock cut slopes on the orientation of planar structures, as described in this report, as a possible interim method that may be used to design rock slopes. However, the fallibility of the method has not as yet been fully established. Much time-consuming work must be done before methods of measuring the effect of planar structures, stress, and other factors affecting the stability of rock slopes are developed and proven.

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Measurement on the Pilot Bore for the Straight Creek Tunnel

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•THE MEASUREMENT of rock loads and ground movements associated with the excavation of underground openings had its beginnings approximately 25 years ago in Europe. Although there are earlier records of salt mine convergence measurements and of load measurements in Alpine tunnels (1920's), all these early measurements suffered from the lack of precision measuring devices. The early salt mine convergence measurements were obtained by determining the time to squeeze tight a steel plug in a drill hole. The Alpine tunnel load measurements were based on the observation of the crushing of wooden blocks placed in timber sets.

The availability of precise measuring devices has permitted a more accurate approach to measuring the response of rock to mining and construction operations. To date, the greatest application of available rock mechanics instrumentation has been in Europe. The reason for this early development in Europe was the need for information related to mining operations under difficult conditions and near populated areas. The problems of underground mining and construction in Europe became critical because of the necessity of making a maximum recovery from limited ore reserves. This problem has not been as severe in the United States or Canada because of the more abundant resources and the less dense population. In general, in the United States it is possible either to buy the surface above a mining operation if damage is anticipated or to forego mining deposits under populated areas.

The ability to measure the reaction of rock masses to mining operations has made it possible for Europeans to develop lower cost mining methods which permit ore recovery within permissible damage limits to surface structures.

Terrametrics, Inc., of Golden, Colorado, measured rock loads and ground movements in relation to the excavation of the Straight Creek Tunnel Pilot Bore. They were specifically measuring:

1. The relationship between the load imposed on the steel sets and the designed load,
2. The zone of influence around the tunnel where the ground is strained (stressed or de-stressed) as the result of the tunnel excavation,
3. The strain variation outward from the tunnel walls resulting from the tunnel excavation, and
4. The rate of decrease of overall ground strain rates with time and tunnel face advance.

Instrumentation stations were placed along the tunnel at intervals sufficient to obtain a statistical approximation of the rock loads and ground movements in relation to various parameters, i. e., rock type, joint spacing, overburden depth, faulting, alteration, and tunnel geometry.

A total of 44 instrumentation stations was installed. Figure 1 is a diagram of one of the primary instrumentation stations (PIS). This type of station is designed to:

1. Measure the rock load on the steel support;
2. Measure the overall radial deformation of the roof and walls of the tunnel; and
3. Permit the calculation of the rate of change of the rock strain rate as the tunnel face advances away from the station.

Paper sponsored by Committee on Soil and Rock Properties and presented at the 45th Annual Meeting.

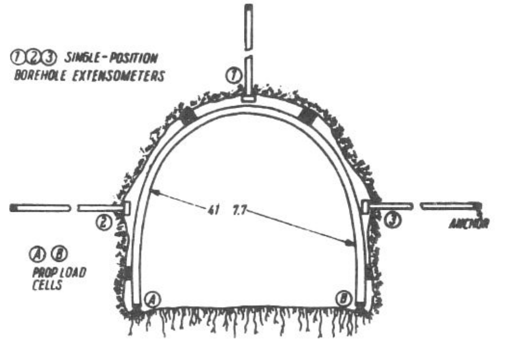

Figure 1. Primary instrumentation station (PIS).

Instrument stations were placed as close as possible behind the advancing tunnel face. The distance varied from 5 to 15 ft. Care was exercised to prevent blast damage to the instruments or their signal wires.

ROCK LOADS

Figure 2 is a typical vertical load history curve for PIS No. 6. These load measurements are directly related to the tunnel geometry, the steel set spacing, and the mass rock density. This permits the calculation of the rock load in pounds per square foot and the indicated rock arch-height in feet. The set spacing and rock density are dependent on the structural characteristics of the rock being excavated. The Straight Creek Pilot Bore is in a terrain of Precambrain granitic and metamorphic rocks.

The load history curve shown in Figure 2 demonstrates one of the phenomena measured in this tunnel—an extremely high-load zone which follows the tunnel face. Similar high loads were measured at every instrument station. The measured peak loads appear to be limited only by the yield strength of the steel or the crushing strength of the timber blocking.

This phenomenon closely follows the measurements made around several instrumented longwall coal faces in Europe. In coal mining, these loads are referred to as the abutment loads.

The apparent abutment load measured at Straight Creek can be explained by a modification of the theory used to describe the deformation of the rock roof behind an advancing longwall coal face. In brief, the hypothesis advanced is as follows (Fig. 3):

1. In between the face and the major rock section overhead is the near-roof beam, which may have considerable height in a nonstratified rock such as at Straight Creek.

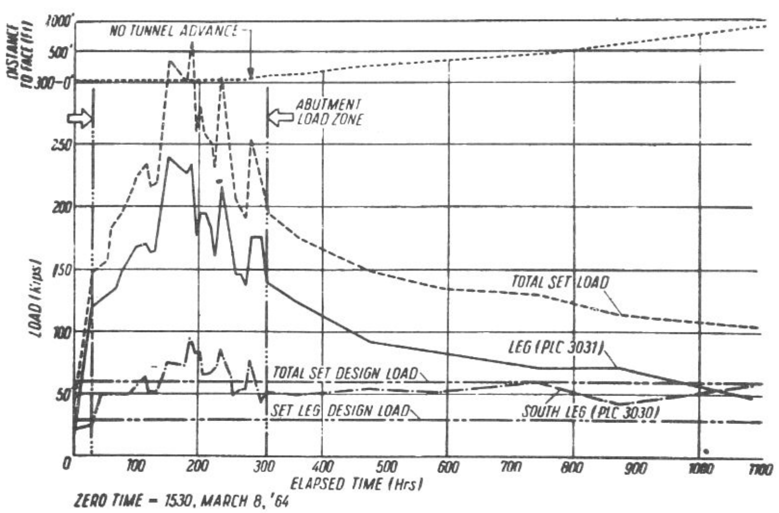

Figure 2. Typical vertical load history curve (PIS No. 6)

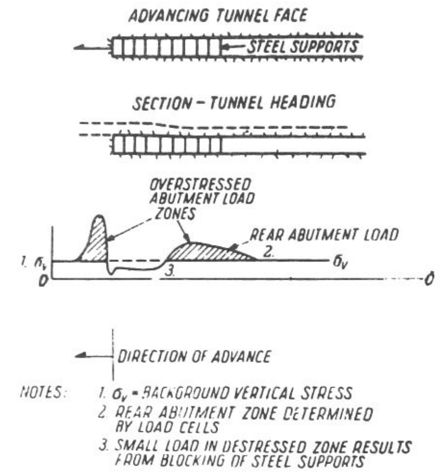

Figure 3. Hypothesis for the loading phenomena.

2. This cantilevered near-roof beam is deflected downward as the tunnel face advances away from the instrumented station.

3. The deflection of the near-roof beam is resisted by the strength of the beam, the tunnel walls, and the steel tunnel supports.

4. The tunnel walls strain in response to the increased load; the near-roof beam deflects; and the steel supports are left attempting to support the vertical load previously supported by the rock excavated for the tunnel.

5. The measured peak loads (abutment loads) result from the attempted support by the steel sets of the total vertical load which is forcing the near-roof beam to deflect and the tunnel walls to strain.

6. The distance from the face of this apparent abutment load is determined by the stiffness or load-carrying capacity of the near-roof beam and tunnel walls, in conjunction with the steel support.

7. The steel support attempts to resist the deflection of the near-beam. The resistance of the steel is determined by the manner in which the blocking loads the particular steel set in question. The manner of the blocking is in turn determined by the geology and the construction technique.

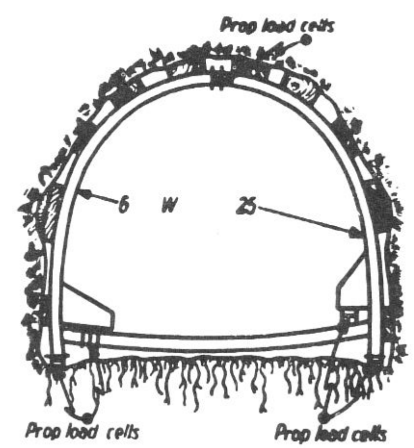

Figure 4. Special instrumented sets.

8. Since the steel tunnel supports cannot support the entire load of the rock except in the case of a tunnel near the surface, either the steel or the blocking must be deformed so that the vertical load can be transferred to the tunnel walls, which are the major load-carrying structural units in any tunnel.

9. Once the rock has deformed sufficiently to establish stable conditions, the temporary set overloads will cease. When the rock deformation ceases, the steel set deformation will also cease. The stable loads measured on the steel tunnel supports are principally from the rock in the tension zone above the tunnel. A possible additional source of a portion of the stable set load may be some residual load from the

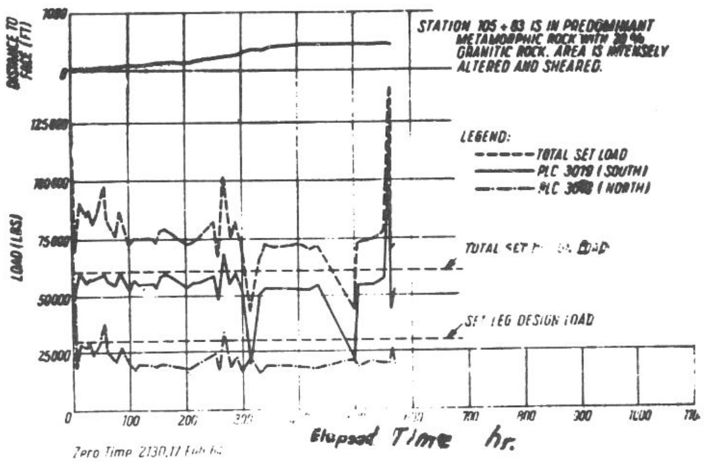

Figure 5. Load history, station 105 + 83: indequate steel size.

squeezed blocking and elastic deformation of the steel. This load should decrease with time as the wooden blocking ages and relieves this applied load.

Special load-measuring stations were installed in the two shear zones encountered in the Straight Creek Tunnel Pilot Bore. These stations were constructed to permit the measurement of the horizontal and vertical steel loads. Figure 4 shows how these special sets were constructed and instrumented. Knowledge of the loads on these special sets has permitted the calculation of a friction factor for sheared and altered material in the major shear zone.

In the sheared zone, the rock loads built up above the steel sets' (417. 7) design value, and thereby deformed excessively, necessitating replacement with a heavier steel

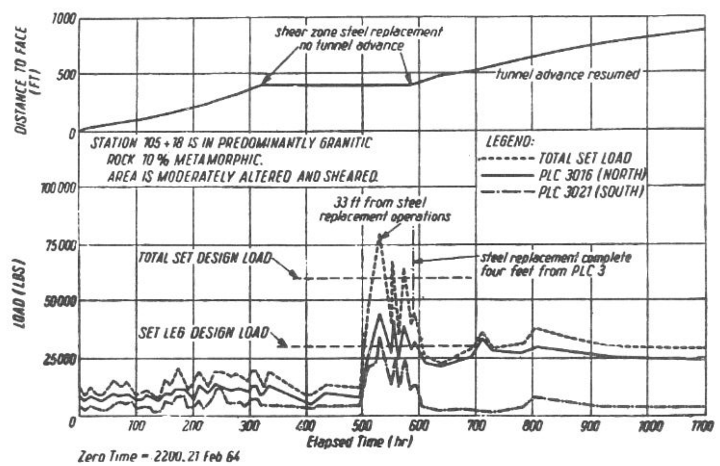

Figure 6. Load history, station 105 + 18: effect of steel replacement.

section (6H 25). The inadequacy of the original steel to support the rock loading in this shear zone can be seen in Figure 5.

When the new steel was substituted for the original lighter steel, the tunnel advance had to be stopped. During the course of replacing the steel supports in the major shear zone, a small amount of rock fell out in the shear zone. This action temporarily raised the loads measured on the instrumented steel sets adjacent to but not in the shear zone. Figure 6 shows the effect of the steel replacement on a nearby load-measuring station just outside the shear zone.

The measured stabilized rock load on the heavier steel after replacement was 6000 psf vertically and 4800 psf horizontally. Figure 7 is a graph of the total vertical and horizontal load histories of the two sets instrumented in this shear zone. The total design load for these two sets is approximately 500, 000 lb. Of particular interest is the instability of the loading and overloading during the first 100 hr after installation.

The selection of steel with a design load of 60, 000 lb appears rather good for this pilot tunnel, in respect to the stabilized loads. It is obviously not sufficient for the shear zone at Straight Creek.

GROUND STRAINS

Figure 8 is a typical borehole extensometer strain rate change plot at a primary instrumentation station (PIS No. 6). It demonstrates initial instability of the rock around the tunnel and the approach of stability with time and tunnel advance. The rock strain stabilized at the same time that the set loading stabilized. This can be seen by comparing Figure 2 with Figure 8.

Figure 8 demonstrates the ground strain (movement) phenomenon, which was typical in the jointed granitic and metamorphic rocks in the pilot bore. Figure 9 is a similar plot for an unsupported granitic tunnel section. Here the joint spacing was from 1 to 3 ft and the strain fluctuation effectively ceased in about 100 hr. The implication was that the joint spacing was the controlling variable in the duration of induced strains around the Straight Creek Tunnel Pilot Bore.

The ground (rock) strains have followed a definite pattern. This pattern appears to be the result of intermittent adjustments of the rock along predefined joint surfaces in response to stress changes associated with tunnel advance. These movements are somewhat erratic, with quiet (low-strain) periods separated by brief strain adjustments

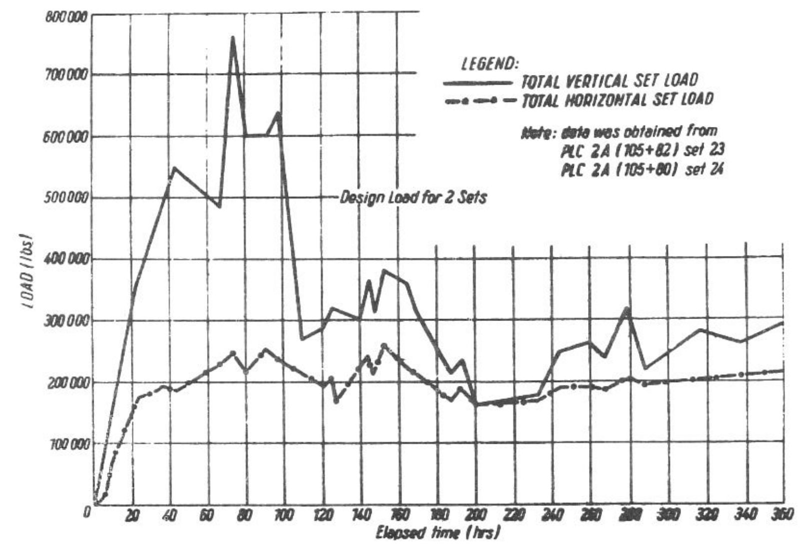

Figure 7. Total vertical and horizontal load histories for two sets in a shear zone.

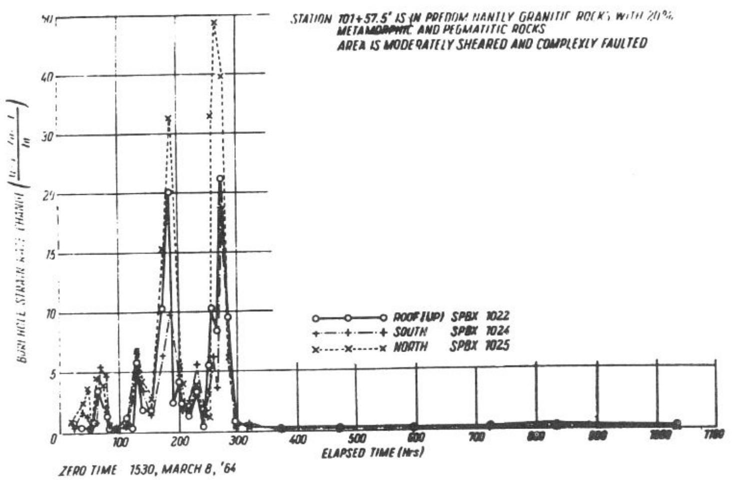

Figure 8. Strain phenomena typical of jointed granitic and metamorphic rocks.

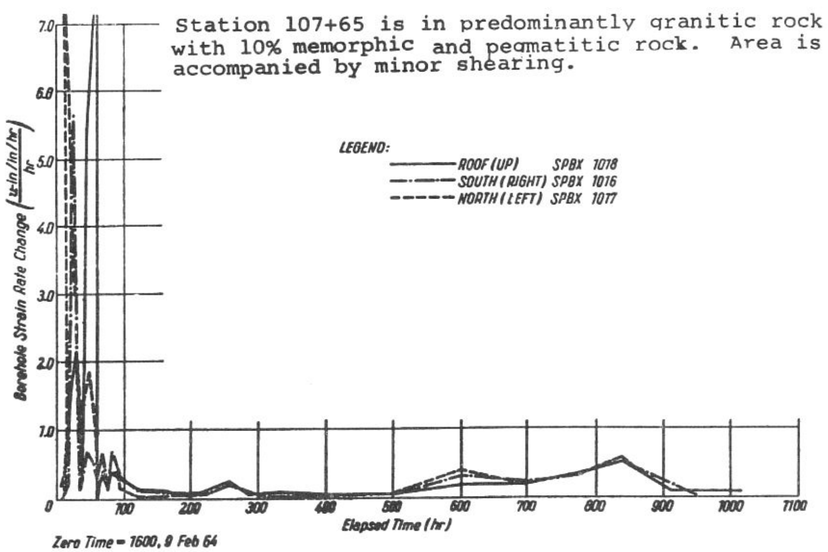

Figure 9. Strain phenomena for an unsupported basically granitic section.

of considerable magnitude. At all the stations instrumented, the magnitude and frequency of these adjustments decreased with time and the advance of the tunnel face away from the station. This pattern of decreasing strain rate is a primary indication of approaching stability.

Figure 10 shows a typical supplementary instrumentation station (SIS). The purpose of this instrumentation, in addition to providing all the information obtained by a primary station, was to permit the determination of the stress change outward from the tunnel in both the upward and the lateral directions.

The ideal elastic approximation of the strain and, therefore, stress distribution around an opening in brittle rocks, has been tested by the strain-measuring instrumentation at the Straight Creek Tunnel Pilot Bore.

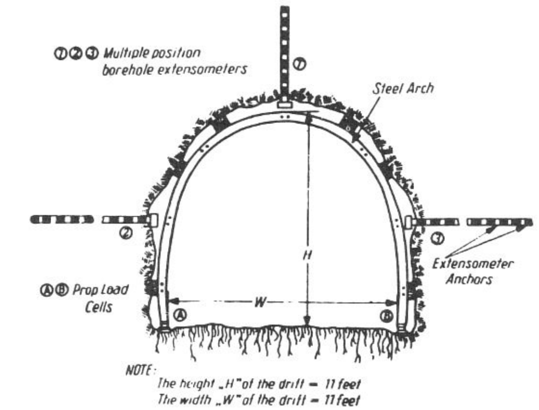

Figure 10. Typical supplementary instrumentation station (SIS).

The most interesting results were obtained from the up-hole instrumentation. Figure 11 is typical of the rock strain variation measured at one station. This figure is a schematic of the relative radial strains measured about the pilot bore. It shows that the general radial strain picture is similar to that anticipated on the basis of elastic theory. This schematic has been derived from curves of relative strain rates for randomly sampled time intervals along the various holes.

One of the immediate applications of this instrumentation is to design a preliminary roof bolting pattern both to support the rock load and to be of sufficient length to penetrate into the compression-optimum anchorage. The unknown is then the joint spacing which influences and controls the roof bolt spacing.

At station SIS No. 2, the indicated bolting pattern would be 6-ft-long bolts (minimum) on 4-ft spacing. The joint spacing was 0.5 to 1 ft, which would require wire mesh or

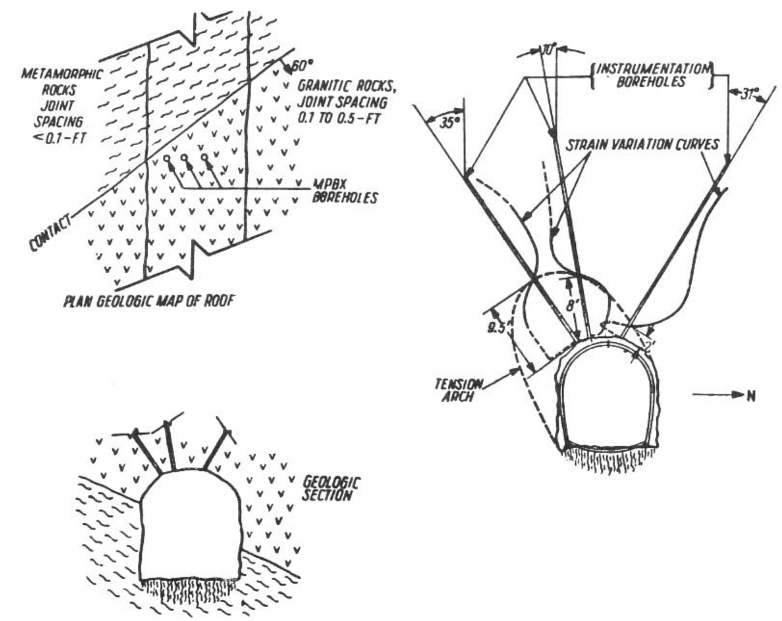

Figure 11. Schematic of the relative radial strains.

chain link fence between bolts. This pattern should both maintain the structural integrity of the tunnel and prevent blocks of rock which may become detached between the bolts from dropping into the tunnel.

CONCLUSIONS

1. The ability to accurately measure loads and rock strains makes it possible to rationally design supports to withstand the applied loads and accommodate the associated strains.
2. The determination of the safety of a rock structure need not be left to a nebulous experience factor. When support loads stabilize and rock strains cease or become negligible, we then have an effectively safe structure.
3. The ability to measure is the major step toward the ability to control.

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Avoiding Damage to Residences from Blasting Vibrations

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This paper presents some basic procedures for avoiding vibration damage to residences when blasting. Particle velocity of the ground near the residence is presented as the most suitable criterion for associating ground vibration with building damage. Two inches per second particle velocity is recommended as a safe vibration level.

A scaled distance, which is defined as distance from the blast to the point in question in feet divided by the square root of the maximum charge per delay in pounds, can be used as a guide for determining the amount of explosives allowable per delay. A scaled distance of 50 $ft/lb^{1/2}$ is offered as a safe limit if no seismic instrumentation is used to measure vibration levels from the blasts. A scaled distance less than 50 $ft/lb^{1/2}$ may be a safe limit when instrumentation has been used to establish vibration levels.

•THE PRIMARY purpose for the use of explosives in drill holes is to break rock. This problem has been studied carefully, and most blasters are well aware of the problems associated with rock breakage. Nevertheless there is a hazard associated with blasting that frequently is overlooked or ignored because it is not useful to the blaster. This hazard is the strong seismic waves generated by the blast that progress through the rock in all directions from the blast area. These seismic waves can adversely affect structures at considerable distances from the blast. Therefore, if damage is to be avoided when blasting, the users of explosives must know the basic fundamentals of the generation and propagation of seismic waves from blasting and the vibration levels of these waves that can be considered nondamaging.

Numerous criteria have been proposed to establish a maximum safe vibration level. Rockwell (5) in 1927 stated that, as a result of his instrumented tests, structures that are farther than 200-300 ft from a blast would not be damaged. He also pointed out the need for measuring vibrations from blasting in order to establish the level of vibration as a function of charge size and distance.

In 1942, Bureau of Mines Bulletin 442 reported particle acceleration as the best criterion for estimating damage to structures (6). For example, a particle acceleration of 0.1 g or less in the structure was labeled no damage, 0.1 to 1 g as caution, and a particle acceleration above 1 g was listed as possible damage.

F.J. Crandell (1) presented, in 1949, a criterion based for the first time on vibration levels in the ground in the vicinity of the structure. This criterion called for an energy ratio, which he defined as acceleration squared divided by the frequency squared. Where acceleration is in ft/sec^2 an energy ratio below 3 was considered safe and above 6 was called the danger area.

Numerous studies were performed and results, in which various criteria were suggested, were published between 1949 and 1960. During this time, particle displacement of 0.03 inches was adopted by several states as a safe blasting limit.

In 1957, Langefors, Kihlstrom, and Westerburg (4) proposed a criterion based on particle velocity in the ground near a structure. In this report, 2.9 ips was listed as producing no damage, but increasing velocities would then begin to produce damage.

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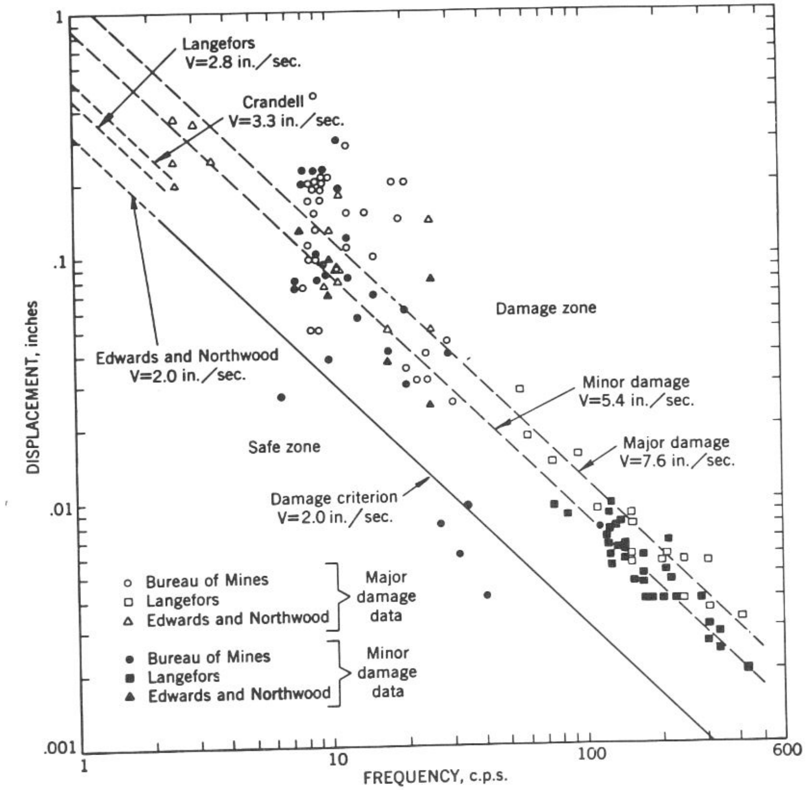

Figure 1. Recommended damage criteria.

Edwards and Northwood's paper (3) also set forth particle velocity as a criterion for damage control; 2 ips was considered safe by these researchers.

In 1959, the Bureau of Mines was requested to reinvestigate the problem of vibrations from blasting and their effect on structures. At that time it was noted that data concerning damage to structures are most difficult to acquire since intentional damage to structures for test purposes is costly, and unintentional damage becomes unavailable by legal proceedings and hostile witnesses. Therefore, one of the first tasks undertaken was a search of the literature and assembly of all published data on buildings that were damaged as a result of blasting vibrations. Of the numerous papers avilable only three reports included data along with the analysis and conclusions. These were the Bureau of Mines Bulletin 442 by Thoenen and Windes (6), the Hydro Electric Commission Report from Canada with Edwards and Northwood as the principal investigators (3), and the report by a group in Sweden under the direction of Langefors (4).

The three sets of data were combined and statistically analyzed tc determine if one criterion could be considered more closely associated with building damage then any of the other criteria. When these combined displacement-frequency data were plotted on log-log coordinates, it became apparent that there were no significant differences among the data. Thus, even though these sets of data were obtained in different countries, by different researchers using varied instrumentation, the data could be pooled and treated as a single group. Figure 1 shows a composite plot of these data. From the statistical analysis of the data, it was concluded by Duvall and Fogelson (2) that damage to residences is proportional to particle velocity and that major damage (fall of plaster, serious cracking) can be expected at a particle velocity of 7.6 ips, minor damage (fine plaster cracks, opening of old cracks) at a particle velocity of 5.4 ips, and that 2 ips appears reasonable as a separation between a relative safe zone and a probable damage

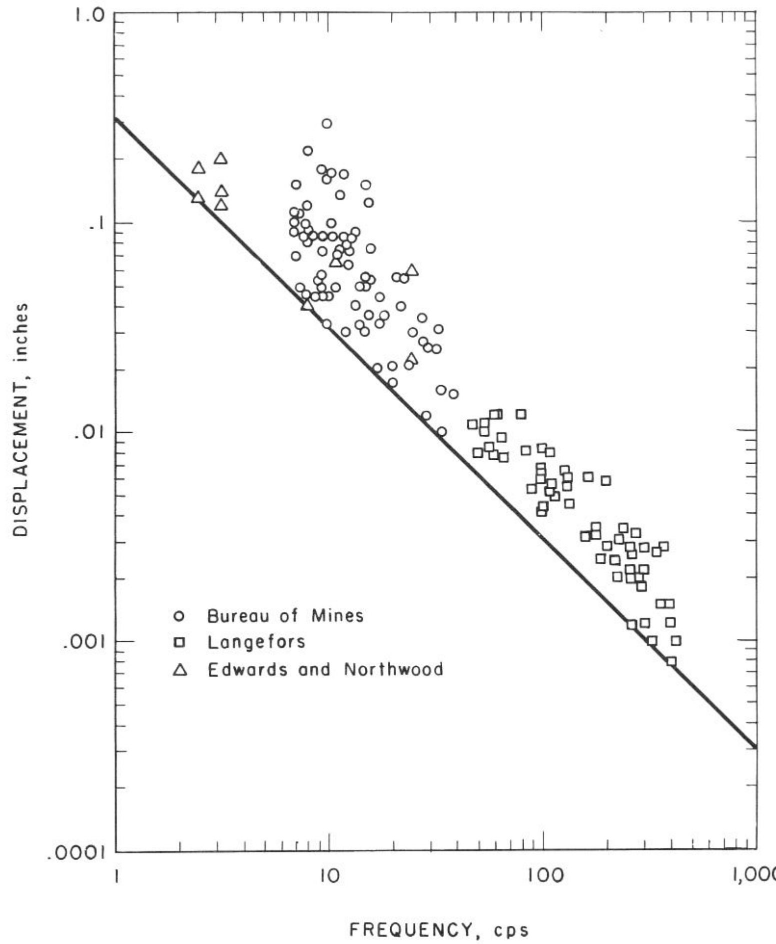

Figure 2. Displacement vs frequency for no damage.

zone. Incidentally, this safe limit agrees with that offered by Edwards and Northwood. This work was presented in a Bureau report and the criterion of 2-ips particle velocity has been generally accepted as a safe vibration level by people working in the field of blasting vibrations.

The fact that 2 ips is reached at a particular structure does not necessarily mean that damage will occur. Many structures and residences have experienced vibration levels greater than 2 ips without any observable damage. As an illustration, Figure 2 shows the 2-ips line through all the no-damage data points above 2 ips that could be found in the published literature.

From these investigations it has been concluded that if one or more of the three mutually perpendicular components (radial, vertical and transverse) of vibration in the ground near the structure have peak particle velocities in excess of 2 ips, there is a fair probability that damage to the structure may occur. Conversely, if the components of vibration in the ground near a structure have peak particle velocities less than 2 ips, there is a low probability that damage to the structure may occur. The probability of damage increases as the vibration level increases above 2 ips and the probability of damage decreases as the vibration level decreases below 2 ips.

With a safe vibration criterion established, the next problem was to determine what variables contributed significantly to the vibration level. Obviously, charge-size and

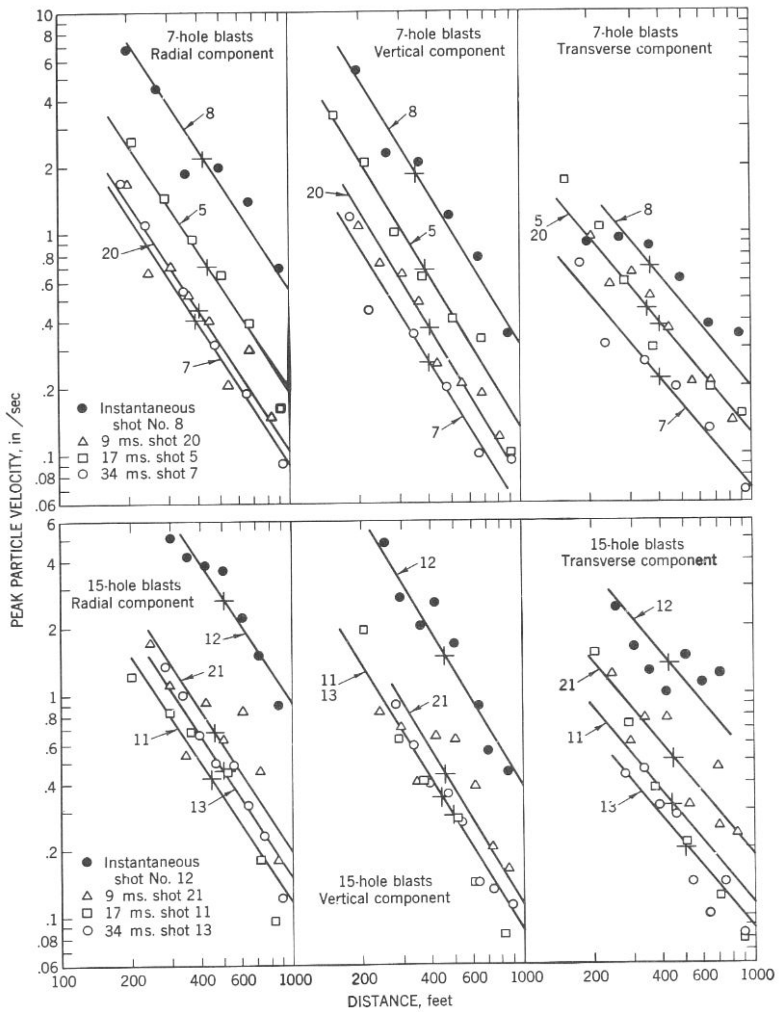

Figure 3. Particle velocity vs distance for 1- and 3-hole quarry blasts.

distance from the blast point to the structure in question are major factors controlling the vibration level. Thus, the development of a usable propagation law governing charge-size and distance became the next project.

The first tests consisted of a series of blasts ranging from 1 hole to 15 holes per blast. Delay intervals varied from zero to 34 milliseconds. The data from these tests plotted by component of velocity (radial, vertical, transverse) are shown in Figures 3 and 4. From a statistical analysis of these tests it was concluded that when a blast consists of several delays, it is the charge weight per delay and not the total charge weight that determines the vibration level. Secondly, the increase in the number of delays does not increase the vibration level. Finally, since straight lines fit the data well when the particle velocity data are plotted as a function of distance on log-log coordinates, an applicable propagation equation can be expressed in the form

$$V = HW^{\alpha} D^{-\beta} \tag{1}$$

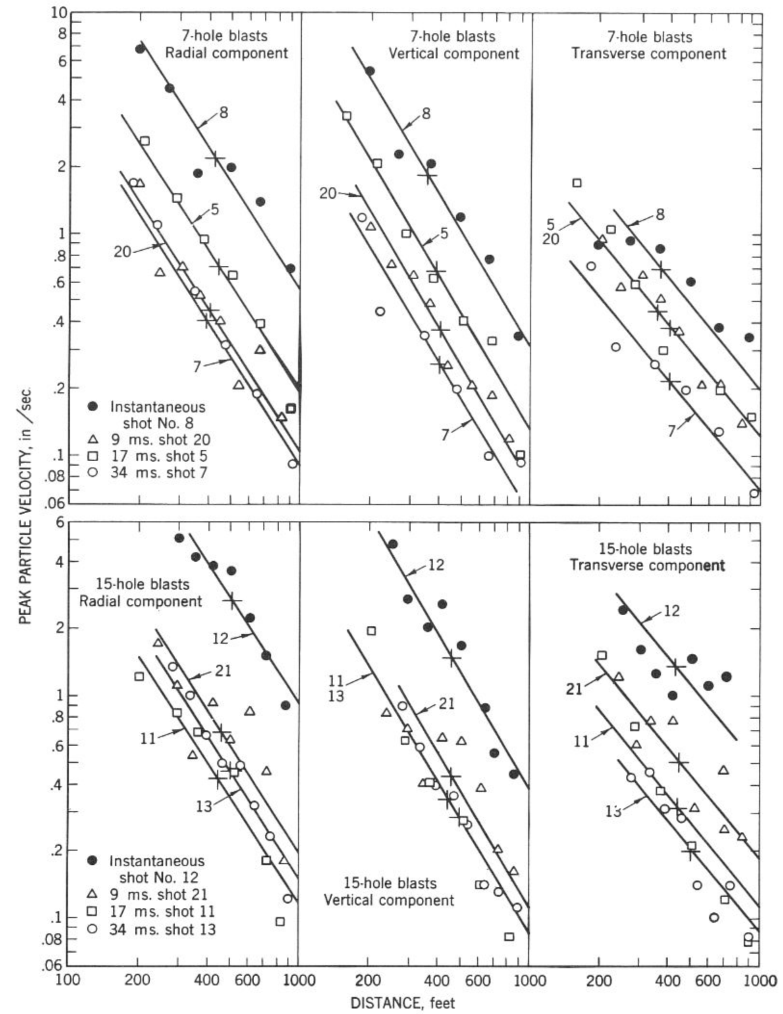

Figure 4. Particle velocity vs distance for 7- and 15-hole blasts.

where

V = particle velocity, ips,
W = maximum charge weight per delay, lb,
D = distance from blast to measurement point, ft,
H = constant for a particular site,
β = exponent of D and the slope of the regression lines in Figures 3 and 4, and
α = exponent of W.

The exponents α and β and constant H must be determined for each site considered.

From further testing at numerous sites it has been concluded that this general propagation equation can be expressed as

$$V = H (D/W^{1/2})^{-\beta} \tag{2}$$

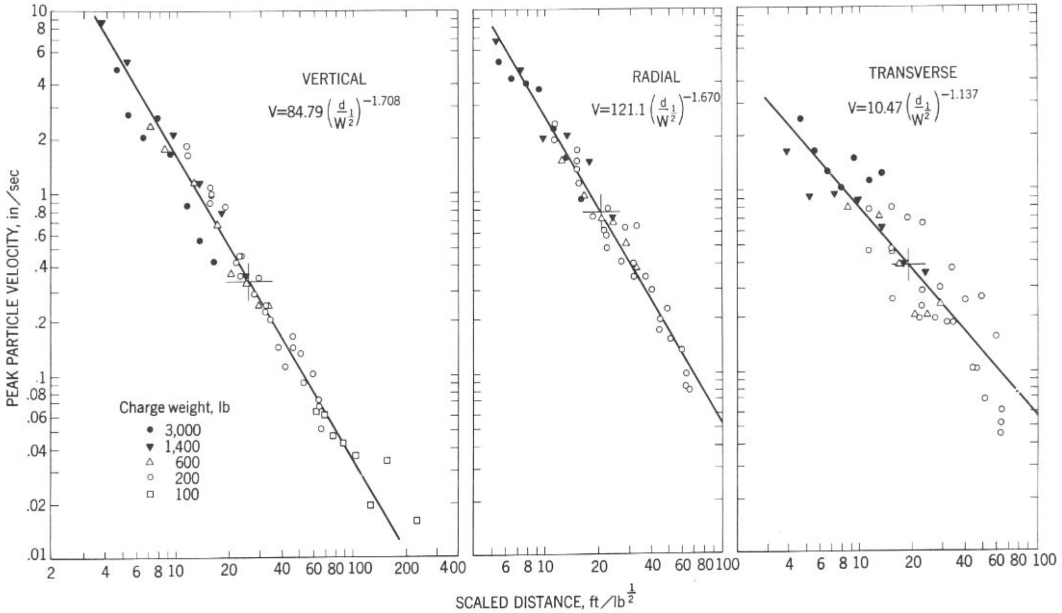

Figure 5. Particle velocity vs scaled distance for instantaneous blasts.

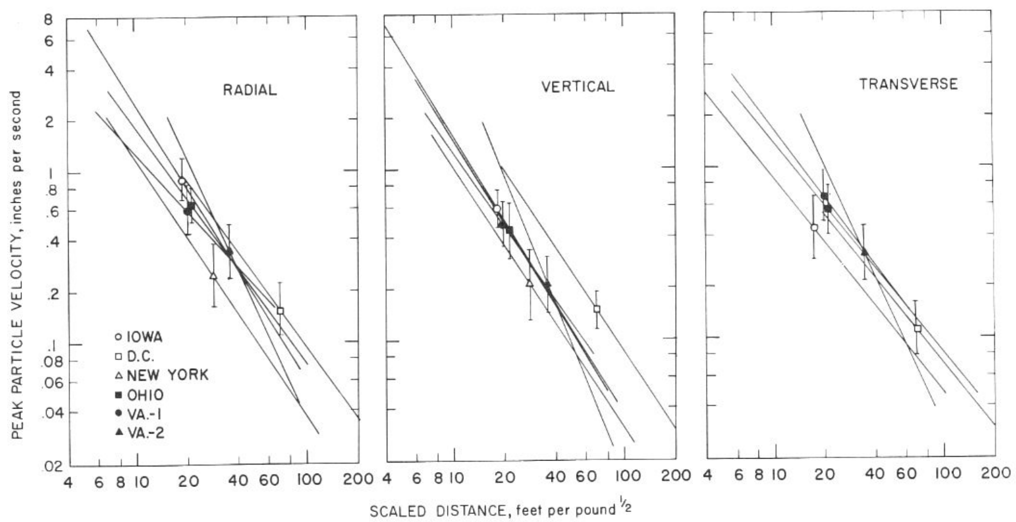

Figure 6. Particle velocity vs scaled distance, combined data.

where

D = distance from blast to measurement point, ft,
W = maximum charge weight per delay, lb, and
H = constant for a particular site.

When particle velocity data from one site are plotted on log-log coordinates as a function of the scaled distance $D/W^{1/2}$, good grouping of the data is

TABLE 1

MAXIMUM CHARGE WEIGHTS PER DELAY ALLOWABLE FOR SCALED DISTANCES OF 50 AND 20 FT/LB$^{1/2}$

Distance (ft)	Maximum Charge Weight Per Delay	
	Scaled Distance (50 lb)	Scaled Distance (20 lb)
100	4	25
500	100	625
1,000	400	2,500
2,000	1,600	10,000

obtained. An example of this grouping is shown in Figure 5. The plots in this figure show the same data as were presented in Figures 3 and 4. It can be seen from this figure that one could select a scaled distance for this particular site at which the safe vibration level would not be exceeded. This scaled distance could then serve as a guide for determining the amount of explosives that could be used without exceeding the safe vibration level.

Figure 6 shows the lines of least squares that have been placed through the particle velocity-scaled distance data from each of 6 quarries. To avoid confusion, each data point is not given, but the midpoint of the data is represented by a symbol. A minimum of 24 data points have been used to determine the slope and level of each of these lines. The length of each line represents the range of the data. The vertical line through each midpoint represents one standard deviation. The variation in the slopes and intercepts of the regression lines indicates that a large site effect is present. Therefore, if one wished to establish a scaled distance where the safe blasting vibration level of 2 ips would not be exceeded, the spread in these data must be considered.

For example, at a scaled distance of 20 ft/lb$^{1/2}$, a point that corresponds to 2 ips would be well outside of the spread in the data for the Iowa site, but would be within the spread in the data for the Va-2 site. Thus, a scaled distance of 20 ft/lb$^{1/2}$ could be used only if tests had been performed at the site in question and the results showed that the particle velocity-scaled distance curve is situated along the left side of the general plots shown in Figure 6. If, however, the data fall along the right side of these plots, a scaled distance of 20 ft/lb$^{1/2}$ could not be used. But a point which corresponds to the safe blasting vibration criterion at a scaled distance of 50 ft/lb$^{1/2}$ is well outside the spread in any of the curves shown and very probably any other quarry. Therefore, a scaled distance of 50 ft/lb$^{1/2}$ would provide a reasonable margin of safety for most quarry operations. But if one suspects that conditions are unique at a particular site, vibration measurements should be taken before establishing a minimum scaled distance as a safety limit. Table 1 gives some charge weights and distances that correspond to scaled distances of 20 ft/lb$^{1/2}$ and 50 ft/lb$^{1/2}$. If one has established that a scaled distance of 20 ft/lb$^{1/2}$ contains a reasonable safety factor, 25 lb of explosives per delay can be detonated and 2 ips will not be exceeded at a distance of 100 ft, the detonation of 625 lb per delay would produce vibrations below 2 ips at 500 ft, and so on. But if no measurements have been taken, a scaled distance of 50 ft/lb$^{1/2}$ should be used. Thus, 4 lb per delay is the maximum safe amount for a distance of 100 ft, 100 lb per delay for a distance of 500 ft, and so on.

There are other variables that can account for the spread in the data shown in Figure 6. At some quarries there is a variation in the propagation law parameters, H and β, with direction from the blast area. Therefore, if a scaled distance less than 50 ft/lb$^{1/2}$ is to be used, measurements should be made in each direction of possible vibration damage.

The type of rock in which the blasting occurs can have an effect on the level of vibrations that result from blasting. Therefore, if the scaled distance from the blast to a potential damage point is less than 50 ft/lb$^{1/2}$, new vibration measurements should be made when blasting operations are moved from one rock type to another.

The method in which the blast is detonated has a significant effect on the resulting vibration levels. For example, if a row of holes is to be detonated with detonating fuse and is initiated somewhere in the middle of the row of blast holes, a distinctly higher level will result than if the blast were initiated at one end. This is because two holes detonate during each delay. Current investigations indicate that blasts detonated with electric caps produce vibration levels that can vary considerably. This variation is probably caused by the time spread in the initiation of a group of caps each with the same nominal delay. But, this spread can only result in a lower vibration level than anticipated.

There is a problem concerning instrumentation when one decides to use particle velocity as a safe vibration. Present seismographs are of the displacement type and instrumentation such as used by the Bureau of Mines, although accurate, is not generally satisfactory for routine measurement. There are no velocity seismographs commercially available at the present time, but several manufacturers have expressed interest in producing such an item. Also, components are presently available if one wishes to assemble a velocity measuring unit. However, peak particle velocity can be obtained from either displacement or acceleration records by differentiation or integration, providing accurate frequencies can be determined.

From the 6 years of quarry vibration investigation by the Bureau of Mines, it has been concluded that the vibration levels are not affected significantly by varying the type of explosives. This applies to the generally available commercial explosives used in the quarry industry.

SUMMARY

Peak particle velocity is considered to be the best criterion for associating ground vibration with building damage, and 2-ips particle velocity is a safe vibration level to avoid vibration damage from blasting.

A scaled distance of 50 ft/lb$^{1/2}$ is considered safe even though instrumentation has not been used at the site in question. Scaled distance is defined as the distance from blast to point in question in feet divided by the square root of the maximum charge weight per delay in pounds.

Scaled distances less than 50 ft/lb$^{1/2}$ may be used as a safe limit if instrumentation has been used to establish the level at these points.

Method of initiation, direction from the blast, rock type, and rock attitude can have an effect on vibration levels.

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