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Chapter 4

INFLUENCE OF BLASTING ON SLOPE STABILITY; STATE-OF-THE-ART

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ABSTRACT

In order to predict the influence of blasting on slopes, one must first understand the action of explosives, the manner in which rock is broken or displaced, and how seismic waves are transmitted, and the nature of these seismic waves. In this paper, the author describes the action of high explosives on rock masses, beginning at the source of the explosion, extending through the zone of rupture to a distance where only elastic waves of low energy persist. The paper describes the relationships between explosives charge quantities and distances, including the character and intensity of the seismic waves in various types of terrain.

In looking at the analysis of slopes subjected to seismic waves, it has been common practice to assume a simplified model in which the anticipated acceleration is applied horizontally to the slope as an equivalent static force, or a model in which there is a rigid base subjected to mechanical shaking. Such models do not accurately portray blasting activity taking place within or near the slope in question. In dealing with blasting effects, it is important to understand seismic wave types, wave lengths, attenuation and transmission paths, as well as vibration intensity.

In most cases, those slopes that are the least stable under static loading will also be the least stable under dynamic loading, although there may be rare exceptions. Thus, the more that is known about a slope statically, the better will be the assessment of it dynamically.

UNDERSTANDING BLASTING PHENOMENA

One cannot proceed very far with a discussion of blasting effects on slopes without recognizing the need for an understanding of the full range of effects, beginning at the source and extending to such distances that only low seismic levels remain. Without an understanding of the various phenomena involved, it is easy to be drawn into inappropriate assumptions about blasting effects (whether ground rupture or shaking) and/or the use of inappropriate methods of stability analysis.

At the second conference in this series, the writer presented a discussion of blasting effects (Oriard, 1971). The writer will not repeat that discussion in detail. However, a brief overview of a few of the previous comments will provide the background for a better understanding of the present paper.

THE BLASTING SOURCE - THE INELASTIC ZONE

It seems useful to discuss the near-source phenomena for at least two reasons. One is for the purpose of controlling breakage. The other is to point out the distinction between vibration effects and those non-vibration, inelastic effects near the source, such as rock rupture, block motion and gas venting. The latter effects have an important bearing on the stability of bench faces and the near-surface portions of excavated slopes.

When an explosive charge is detonated in a borehole in rock, a high-pressure shock wave is transmitted to the rock, followed by a longer-acting pressure under the action of the expanding gases. The shock wave develops a very high pressure, capable of crushing the rock for a distance of the order of 1 to 3 charge radii, but is dissipated very quickly. The propagating pressure pulse develops radial cracks around the borehole, and these are further advanced by the continuing expansion of the explosives gases. Although a larger number of cracks may begin at the perimeter of the crushed zone, it is common for a group of some 8 to 12 cracks to become more prominent and extend to greater distances than the others. According to the principles of fracture mechanics, less energy is consumed in extending the more prominent, existing cracks than to develop new ones. Under the action of the initial stress waves, additional cracks also develop at the locations of flaws within the rock mass, because these flaws provide points of stress concentration.

There was a considerable amount of research done on various aspects of explosives action on rock during the 1950's and 1960's. A limited amount of such research continues at certain universities which have special interests in the subject,e.g. U. of Maryland (Fourney and Barker, 1979), others. In general, this on-going research tends to confirm the basic concepts developed during earlier investigations and construction experience, with certain refinements being advanced as research continues.

The amount of rock directly ruptured, fractured or displaced by the blast is a function of a number of different variables involved in the blast design. Some of the more important, (in addition to the characteristics of the rock mass itself) are:

1. Charge size.
2. Charge concentration or charge spatial distribution.
3. Type of explosive.
4. Depth of burial (distance to any free surface).
5. Coupling to the rock.
6. Sequence and timing intervals of detonation of multiple charges.

Trends in Blasting Technology

In recent years, the principal directions of expanding blasting technology have been found at opposite ends of the spectrum. On the one hand, there has been an increase in very large blasts and large-scale operations, including explosive excavation and cratering technology. Several countries have shown an interest in ejecta dam feasibility (casting ejecta across a valley by blasting) (Oriard, 1976). On the other hand, delicate excavations are more common, also, including refinements of conventional perimeter blasting technology and new developments in fracture-control blasting (Oriard, 1981).

It seems likely that the earliest concepts of blasting were developed around cratering principles, then modified for bench blasting as more sophisticated drilling equipment evolved. Every few years or so there seems to be an interest in reviving the cratering concepts, or expanding on them as they might apply to bench blasting or other types of blasting. In their simplest forms, the cratering principles apply more directly to the case of heavy, concentrated charges (ideally, the point-charge concept), whereas bench-blasting concepts were developed around the use of long cylindrical charges whose lengths are very great compared to their diameters. The debate is somewhat academic because of the great need to tailor any concept very precisely to the specific conditions encountered at the particular site involved. It makes little difference what name is given to a method. It is always important to place the right amount of explosive at the right location to accomplish the work.

The main attraction of a "method" is to simplify the procedures of blast design. Although this is understandably attractive, it has a tendency to develop wrong practices. It is a deception to consider that the geological world is uniform and that formulated approaches are better than site-specific designs. The explosives engineer would develop better skills if he were constantly honing them to the specific conditions of the site. Formulated approaches should be used chiefly to prevent the first test of an inexperienced person from being disastrously designed.

Cratering and Bench Blasting

Single, concentrated charges placed below a horizontal ground surface will form a crater if detonated sufficiently close to the surface. A flat, shallow crater is formed if there is a shallow depth of burial. The crater becomes larger in volume as the depth of the charge burial increases, but beyond the optimum depth of burial, a decreasingly smaller crater is broken, and the rock is merely fractured, not ejected. At great depth of burial, the surface remains undisturbed, although there is a zone of crushing and fracturing around the charge location. In order to take charge size into account in such cratering experiments, it is customary to scale the depth of burial, and the crater volume, by the cube root of the charge weight, based on empirical data that tend to show such a correlation. For example, if we wish to double the radius of a true crater developed by the detonation of a point charge, we will have to increase the charge weight by the cube of 2, or 8 times. The apparent crater, that is left open after the blast, does not precisely fit this cube root law because of ejected material that falls back into the crater through gravity action. According to the above, then, the "scaled depth of burial" of a point charge $(dob)_{pt}$ is:

$$(dob)_{pt} = \frac{DOB}{W^{1/3}} \quad \text{(by definition).}$$

Cratering principles can be applied to explosives excavation (in which material is purposely ejected or excavated directly by the explosives action itself), or various forms of conventional mining and excavating operations in lieu of the more common bench-blasting principles. Of course, typical cratering calculations do not apply to charges near a vertical free face, unless modified suitably. For example, a long column of explosives near a free face, and parallel to the face, could be said to represent a linear crater charge with a depth of burial equal to the distance to the free face, with the expectation of little rock breakage beyond the charge (that is, "below" the crater).

Using cratering principles, one can expand the concept of a point charge to two dimenions for line charges, and to three dimensions for array charges, using simple dimensional analysis. In the case of a line charge, an additional dimension is being added. The unit weight of the charge now becomes the charge weight (W) divided by the charge length (S), or (W/S). Similarly, the unit weight of a plane (array) charge is the charge weight divided by the area of the plane. In a square array, each side of the plane could be called (S) and the area of the plane called $(S)^2$. Therefore, the unit weight of the charge is (W/S^2).

Equivalent expressions for scaled depths of burial for point charges, line (row) charges, and plane (array) charges are then:

$$\text{Point charge:} \quad (dob)_{pt} = \frac{DOB}{W^{1/3}}$$

$$\text{Row charge:} \quad (dob)_{ln} = \frac{DOB}{(W/S)^{1/2}}$$

$$\text{Array charge:} \quad (dob)_{pl} = \frac{DOB}{(W/S^2)^{1/1}}$$

Note that as a linear dimension is added to a point charge to produce a line charge, the equivalent scaling changes from the 1/3 to the 1/2 power; and as an additional dimension is added to produce a plane charge, the equivalent scaling changes from the 1/2 power to the 1/1 power. Therefore, in order to make a line charge equivalent to a point charge, we must take the 2/3 power of it. And, in order to make a plane charge equivalent to a point charge, we must take the 1/3 power of it. If the reader were to manipulate the numbers in accord with the above statements, he would discover a rule that is in agreement with field experience: for an increasing number of simultaneously detonating charges (point to row to array), one should increase the depth of burial for an equivalent mounding of the rock.

The important thing to remember is that there is no "theory" to determine the correct depth of burial. It is determined empirically, by trial and error in the field. It is highly site specific. A scaled depth of burial of 3.0 may produce ideal mounding at one site, yet be inappropriate at another site. The usefulnees of the above relationships comes in reducing the number of trials needed for the design of full-scale operations.

Similar rules of thumb and general basic principles apply to bench blasting. In both concepts, it is a question of acquiring enough field experience in different geologic settings to discover emerging guide lines. For example, it is common in bench blasting to use a spacing-to-burden ratio of the order of 2.0. It is common to use a depth of stemming of the order of 20 to 25 times the hole diameter, or in the range of 0.7 to 1.0 times the hole spacing, and it is also common to drill below the expected depth of excavation an amount which is of the order of 0.3 times the hole spacing. However, these "rules" are highly site specific. At any and all sites, it is essential to determine by observation whether or not these designs produce the desired results, and to make whatever changes are necessary to meet the project demands. Such "rules" are designed mainly to help the inexperienced user of explosives reduce the number of field trials to optimize his results.

Direct Damage From Blasting

The primary interest of this paper is that of blasting effects on excavation perimeters and nearby slopes. From the above comments, the reader can appreciate the difficulty in providing a concise rule to enable him to predict accurately the distance into a slope or final excavation perimeter that fractures might extend for any given size of explosives charge. Too much depends on other features of the blast design besides charge size, in addition to the rock characteristics. As an illustration, let us assume a relatively large charge of explosives, say 450 kg in a single hole. Such a charge would have the capability of mounding perhaps 750 cubic meters of "average" rock. If the charge were a large-diameter crater charge (no bench face), the rock might be broken for about 10 to 15 meters in all directions. That action would be damaging if the charge were detonated against a final wall. However, if the charge were detonated as part of a bench blast, with a free bench face several meters away, the breakage would be asymmetrical. There would be strong movement toward the free face and reduced breakage into the final wall. It could be conceived as a linear crater charge turned 90 degrees, so that the bottom of the linear crater becomes the final wall. We can carry the concept farther by extending the depth of the holes, reducing the charge diameter and trimming a small burden of rock with very little damage to the final wall. Carried even farther, this plan evolves into cautious, controlled perimeter blasting, such as preshearing or smooth blasting. (See Figure 1.)

Direct rock damage of the type discussed above is not merely the fragmentation of rock due to passing stress waves. One of the most important physical effects occurring near the blasting source is that of block motion or inelastic ground displacement, just beyond the zone of fragmentation. Typically, the maximum range of such inelastic displacement will be the result of the venting of explosives gases beyond the immediate crater zone, and not the result of vibration. Identifying the true nature of such disturbance is important for the reason that the methods for eliminating it depend very strongly on what is causing it. Too often there is an automatic conclusion that ground displacement or block motion beyond the immediate crater zone is the result of vibration when usually it is not.

Control of Rock Breakage

Control of rock breakage usually refers to the control of the perimeter of the excavation. Greater control means a smoother, less disturbed final surface. Of course, the word "control" can be used with other meanings, such as fragmentation control, control of the movement of displaced or ejected rock, or control of vibration. If any misunderstanding is possible, a writer should specify his meaning with additional comment for clarification.

To achieve control of the limits of a rock excavation, the explosives user must ensure that the spatial distribution of the explosives is proper for the soundness and smoothness of the final surface that is desired. An example of relatively uncontrolled blasting would be the use of large, concentrated charges, widely spaced. Such blasting will produce an irregular perimeter. Perhaps the least controlled of all would be a single cratering charge. As a general principle, one could say that the least control is achieved with the smallest number of largest concentrated charges; whereas the greatest control is achieved with the largest number of smallest , spatially distributed charges (Oriard, 1971). The accompanying graph portrays

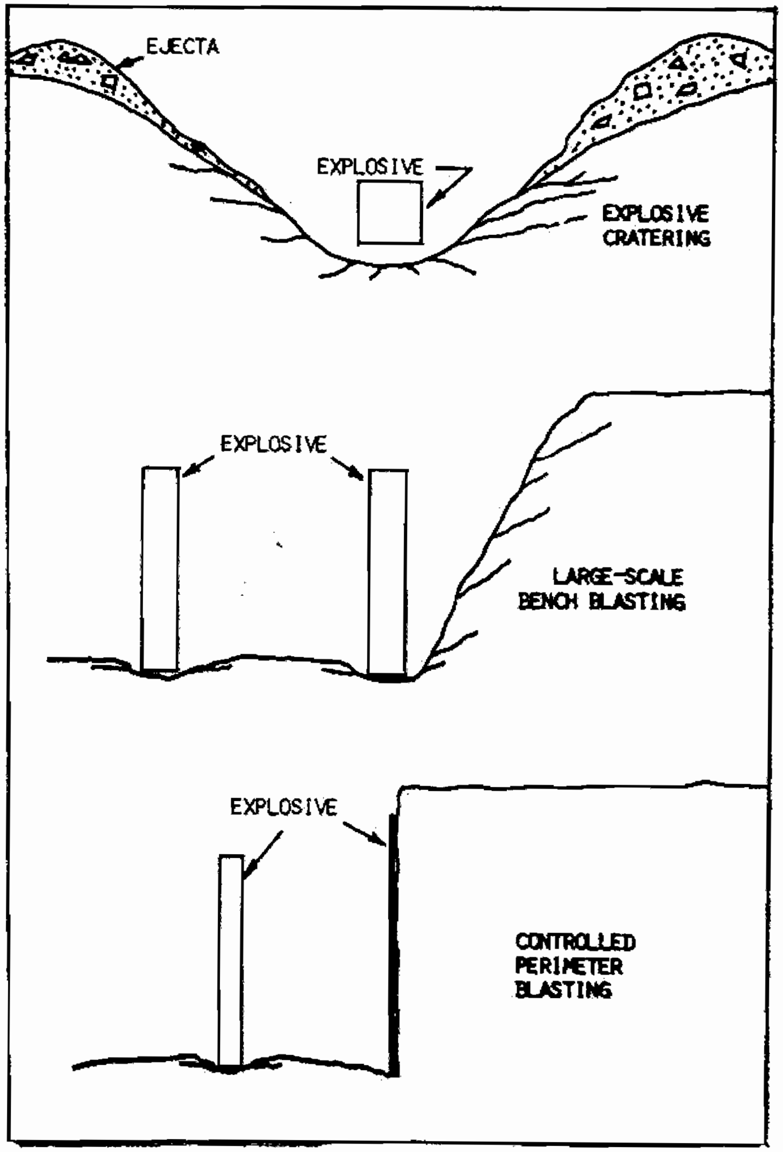


Figure 1 - Illustration of Effects of Spatial Distribution of Explosives Charges

this concept in graphical form. (Figure 2.)

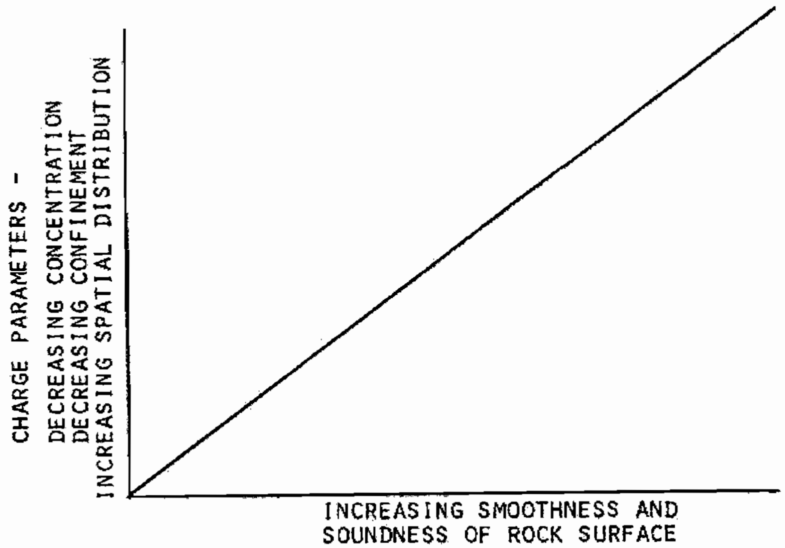


Figure 2 - General Relation of Charge Parameters to Soundness of Rock Surface

Pre-splitting or pre-shearing is a very cautious technique for the control of blast effects at the perimeter of an excavation. With this technique, small-diameter cartridges are detonated in a larger-diameter hole to decouple the charge from the rock surface and enhance the generation of a prominent fracture between the holes, while reducing the development of cracks in other directions (Oriard, op. sit.) In the pre-splitting method, the perimeter charges are detonated first, sometimes as a completely separate operation. If the perimeter charges aredetonated last in the blasting sequence, the method is usually called cushion blasting, smooth blasting, or merely trim blasting. Field experiences gained since the time of the last conference in this series has shown that various modifications of controlled perimeter blasting have proved to be worthwhile in preserving the integrity of open-pit slopes, and the practice is becoming widespread. It has been standard practice in structural excavations for a very long time. Of course, the appropriate extent of such an effort is highly site-specific, depending not only on the characteristics of the rock but also on the needs of the project. In general, the writer prefers a blast design which provides a three-stage approach to slope control. This design includes different details for (1) the pattern holes, (2) a buffer zone consisting of at least one row of holes between the main pattern and the perimeter, and (3) a line of perimeter holes, with increasing caution in the blasting design as the perimeter is approached. Such a design will

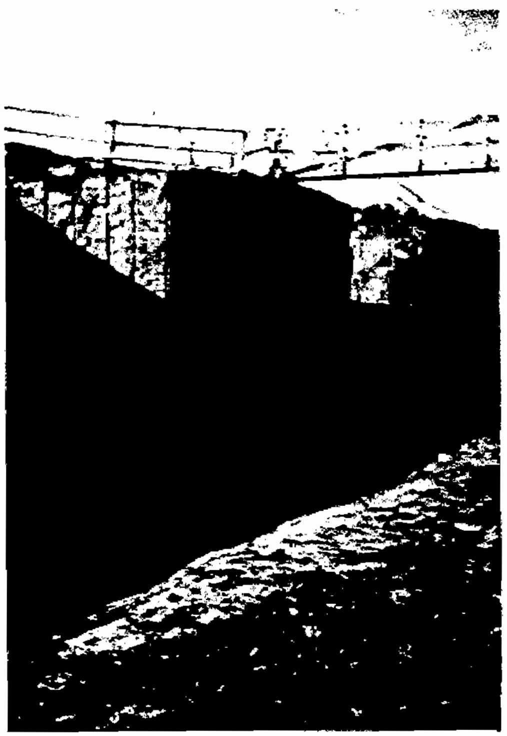
help to preserve the bench faces in a large excavation.

Normally, the writer prefers that the perimeter row be detor last in the detonation sequence, rather than as a pre-split blá When the perimeter is detonated first, as a pre-splitting operá the nearby buffer holes must be placed closer to the perimeter detonated with heavier charges, in order to break and eject all back to the final perimeter surfaces. Thus, the buffer holes n be very precisely drilled and loaded, in order to avoid either break or overbreak. On the other hand, if the perimeter holes detonated last, their function is not merely to generate a frac plane, but to displace rock as well. Thus, they perform a work tion which permits the buffer holes to be farther away and less ically designed. Of course, the work function is a cost saving well.

If financial considerations permit the use of highly control blasting techniques, it is possible to produce complex, "sculpt structural excavations. For example, the photo (Figure 3) sh excavation in which two adjacent openings were blasted in weak leaving an undisturbed, narrow web between the two zones where blasting took place. Although this particular excavation recei attention for the unusually precise sculpture blasting that was it did not involve any new theory, - merely a highly site-speci application of existing technology. The underlining is added t emphasize for the reader the importance of fitting explosives t nology to the specific site conditions involved.

Perhaps the reader will be interested in some recent develop that have application to control of perimeter fracturing, as we other special uses. The method is called fracture-control blas With this method, the sides of the blast holes are notched long inally. The notches or grooves provide stress concentrations w promote fracturing at lower borehole pressures, and control the rection of fracturing, - being primarily within the plane of th notches. The experience of the writer in field tests suggests there can be a reduction in explosives charge quantities of the of 2 to 5 times (Oriard, 1981). The boreholes can be notched w either mechanical tools or high-pressure water jets. Of course either of these notching methods adds time and expense to the p which must be justified on the basis of improved results. Howe it appears that at least two additional options are worth inves ing. One of these would be the development of a drilling tool has the capability of drilling and notching in a single pass. R is currently underway at the University of Maryland regarding tl development of such a tool (Ravinder et al, 1980).

Another option is to make use of a linear shaped charge of e: ives rather than the grooves along the borehole wall. The effe ness of either of these methods would be highly dependent on th jointing characteristics between widely-spaced boreholes. In tl case of large-diameter boreholes and large-scale blasting operat



These photos illustrate the precision that can be achieved when site-specific blasting designs are used for sculpture blasting in structural excavations, even in this poor-quality rock.
Note the undisturbed 40 cm web of rock between the two deeper excavations in the top photo.
The bottom photo shows several undisturbed complex rock monoliths on the far side of a trench which extends to a depth of nearly 15 meters.

Figure 3 - Site Specific Controlled Blasting.

in an open-pit mine, the latter option may be more attractive, at least within the framework of existing technology.

GROUND VIBRATIONS FROM BLASTING

If our interest were only that of predicting the intensity of ground vibrations from blasting, it would not be necessary to discuss wave types and propagation phenomena to any great extent. The subject may be approached empirically, with the inclusion of prediction formulae such as those of the writer, shown later. However, it is necessary for an understanding of the present topic to include sufficient detail of seismic wave phenomena to demonstrate how these relate to slope stability. (A portion of this discussion will appear in a forthcoming AIME volume on underground mining, - see Oriard, 1981A).

After the primary shock front or pressure pulse has passed beyond the zone in which shattering or fracturing of the rock occurs, it passes through the rock in the form of elastic waves or vibrations. As this energy passes through the rock, it takes on different forms which travel at different velocities and cause different types of deformation to occur in the rock. The fastest traveling wave was originally given the name "Primary" or P-wave. This is a compressional wave, sometimes called a radial or longitudinal wave, because the rock is deformed in the radial direction from the energy source. Following the P-wave is a slower traveling wave which was originally called a "Secondary" wave or S-wave. This is a shear wave, sometimes called a transverse wave. Although this wave travels in the same direction as the P-wave, the deformation of the rock is at right angles (transverse) to the direction of the wave travel. The P-wave and S-wave move through the main mass of the rock and have the general name "body waves".

When the body waves arrive at the ground surface, new waves are generated. Some continue through the body of the rock mass as new body waves. Another group travels along the surface and given the name "surface waves". Their motion is quite different from that of the body waves, being characterized by larger amplitudes, lower frequencies, and a lower propagation velocity. In most cases, these waves contain significantly more energy than the body waves, although they do not exist in most underground situations.

If one makes the usual assumption that there is an elastic half space that is homogeneous and iostropic, elastic wave theory describes the wave motions that can be anticipated. In practice, it is simpler and more reliable to determine particle motions by means of field measurements rather than through theoretical calculations. However, it is important to remember that the different wave forms are characterized by different particle motions and are propagated at different velocities. The compressional or dilational wave is propagated with the velocity

$$c_p = |E(1-\mu)/\rho(1-2\mu)(1+\mu)|^{1/2}$$
$$= |(\lambda+2G)/\rho|^{1/2}$$

where $$\lambda = \frac{\mu E}{(1+\mu)(1-2\mu)}$$

and $$G = \frac{E}{2(1+\mu)}$$

E is the modulus of elasticity, ρ is mass density, and μ is known as Poisson's ratio. The constants λ and G are known as Lame's constants. G is also known as the shear modulus.

Compressional wave transmission (propagation) velocities for most rock types fall in the range of about 1500 mps to about 6000 mps, - correspondingly less for weathered or decomposed rock. Most soils fall in the range of about 150 mps to about 1200 mps.

The shear wave propagates at the velocity

$$c_s = \sqrt{G/\rho}$$
$$= |E/2\rho(1+\mu)|^{1/2}$$

The ratio of compressional and shear velocities is

$$c_p/c_s = |2(1-\mu)/(1-2\mu)|^{1/2}$$

Poisson's ratio for most rock materials is very nearly 0.25. Thus, the velocity ratio c_p/c_s is often very nearly $\sqrt{3} = 1.73$.

The Rayleigh wave is named after Lord Rayleigh who was the first to examine the case of this seismic wave traveling along the boundary of a free surface. This wave is characterized by particle motion that is polarized in a vertical plane parallel to the direction of the wave propagation, and the particle motion is elliptical retrograde. When Poisson's ratio is equal to 0.25, the velocity of the Rayleigh wave is 0.92 times the velocity of the shear wave.

Not only do these different wave forms travel at different velocities, but they have the additional characteristic of attenuating at different rates. In the case of spherical symmetry in a nondispersive medium, such as the outward-advancing body wave, elastic theory shows that the amplitude is inversely proportional to the distance. In contrast, surface waves have an amplitude that is inversely proportional to the square root of the distance. Thus, when the point of observation is close to the energy source, there will be a complex combination of several different wave forms. However, as one moves farther from the source, the wave forms become separated, arriving at different times and producing different types of particle motion. The

more distant the point of observation is from the source, the more prominent will the surface waves be compared to the body waves. (There are other types of surface waves in additional to Rayleigh waves, but Rayleigh waves are usually the most prominent).

Both theory and observation suggest that the particle motion transmitted to a free surface is more prominent than for the same wave within the body of the solid. For a wave arriving at normal incidence to a plane surface, the particle amplitude may be doubled. This is of interest to seismic body waves passing through a hillside to an opposite slope.

Theoretically, the stress generated by the passage of a seismic wave is proportional to the product of the acoustical impedance and the particle velocity. When a plane wave arrives at normal incidence to a plane boundary, the partitioning of energy between transmitted and reflected stresses takes place according to the relationship between the acoustical impedances of the two materials, as

$$\frac{\sigma_t}{\sigma_r} = \frac{2\rho_2 c_2}{\rho_2 c_2 - \rho_1 c_1}$$

Kinematics of Particle Motion

The displacement or amplitude of the ground wave is the distance from a particle at rest to its peak or trough as the wave passes. Typical displacements for blasting vibrations of interest fall in the range from about 0.025 to about 2.5 mm. The term amplitude is used also to refer to the trace amplitude on the seismogram (recording of the motion), and can, therefore be somewhat ambiguous.

The frequency of a vibration is the number of cycles that pass a given point in unit time, usually expressed as cycles per second or hertz. Frequencies of interest for blasting usually fall in the range of 1 to 500 Hz, most often being 10 to 100 Hz. Period is the inverse of frequency, and defines the length of time required for one complete cycle of vibration.

Particle velocity is the time rate of change of particle displacement. It is the velocity of the motion of a particle during the passage of the seismic wave beneath the particle. Particle velocity is not the same as propagation velocity. Propagation velocity, or transmission velocity, is the velocity with which a wave travels through a given medium. The propagation velocity varies widely according to the elastic properties of the medium, whereas particle velocity is a function of the vibration itensity. In the following discussion relative to vibration intensity, we will be discussing particle velocity.

Acceleration is the time rate of change of particle velocity. It refers to the acceleration of a particle as the seismic wave passes beneath this particle. For simple harmonic motion, the following relationships apply:

Defining: "x" is displacement at time "t"
"A" is maximum value of x which is equal to the zero-to-peak amplitude
"f" is the frequency
"v" is the particle velocity
"a" is acceleration
"w" is angular frequency

Then:

$$v = 2\pi fA$$

$$a = 0.1\ f^2 A \text{ (approx.) in gravity units.}$$

$$w = 2\pi f$$

$$x = A \sin wt$$

$$v = w \cos wt = w \sin wt + \pi/2$$

$$a = -w^2 A \sin wt = w^2 A \sin (wt + \pi)$$

$$a = 4\pi^2 f^2 A$$

Predicting the Vibration

In order to predict the intensity of ground vibration from blasting, one must consider the influence of the blasting parameters and the influence of the geological setting. If we were always dealing with simple point charges of a single explosives type, the first question would be a relatively simple matter of data scaling, and the second would be a question of wave attenuation, though by no means simple. In reality, the two questions are often closely intertwined because of many departures from the ideal assumptions that are often made to simplify calculations.

In order to compare blasts of different sizes at different distances, it is customary to scale the distance by some function of the explosives charge weight per delay (the amount detonating at any given instant of time), so that such diverse data can be plotted on a simple graph. If the charges were spherical, theory would dictate the use of cube root scaling, because the charge weight would vary as the cube of the radius of the sphere. Dimensional analysis has also been used to support the concept of cube root scaling, but dimensional analysis does not apply to multi-form wave propagation questions. If the charges were long cylinders, we would expect that the use of square root scaling would apply, because the charge weight would vary as the square of the radius of the cylinder. However, in the vast majority of cases involving blasting, neither of these ideal models is accurately duplicated in the field. Many times, the charge weight is increased merely by increasing the number of separate charges, although they are usually long cylinders of explosives. Thus, there are many geometrical complications involved in the question of

data scaling. Statistical analysis will often show variations in the best fit for any of the scaling laws. However, the question is only of academic importance if the decision maker knows where a particular data point falls in the general range of experiences, and understands the consequences of scaling up or down the sizes of charges, or distances, using different scaling laws. There is a considerable amount of scatter in data points relating to blasting vibrations. If the investigator recognizes that a particular vibration were unusually low, he could anticipate that another test of the same design might give a considerably higher value the next time. Also, if he uses a small charge measured at close range to serve as a model for a much larger blast later, at the same scaled distance, he should be aware of the influence his selection of data scaling will have on his final prediction. There will be an important difference in his prediction according to his scaling methods. If he is not aware of the differences, he should try different methods, in accord with his experiences and the conservatism he wishes to incorporate into his predictions (Hendron and Oriard, 1972).

The writer has found it convenient to use square root scaling for prediction of the widest range of blasting conditions. His experience has shown this scaling method to more accurately portray a larger range of field conditions and blasting techniques than cube root scaling or other scaling. The writer has analyzed several hundred thousand vibrations from blasting, and has found the graph (Figure 4) of particle velocity versus scaled distance to fairly represent that experience. The relationships can be expressed mathematically in the form

$$V = H\ (D/W^{1/2})^{-1.6}\ k_1, k_2, k_3, \ldots$$

where
V = peak particle velocity
H = velocity intercept at unity scaled distance
W = charge weight per delay
k factors represent the variations in explosives, confinement, spatial distribution, geology and other parameters of interest.

For a typical upper bound prediction line

$$V = 242\ (D/W^{1/2})^{-1.6} \times (1.0)$$

The slope of (-1.6) represents the attenuation. It is not the same at all sites nor the same for all wave types, but in fact it is surprising how well this slope accurately represents most situations. The attenuation of blasting vibrations with distance is a complex function of the strain level, the various wave forms, and the geological setting. There are several theoretical reasons why one should anticipate a two-slope attenuation curve, rather than the straight line shown in the prediction curves of this writer. For waves which begin at very high strain levels, we should anticipate a more rapid

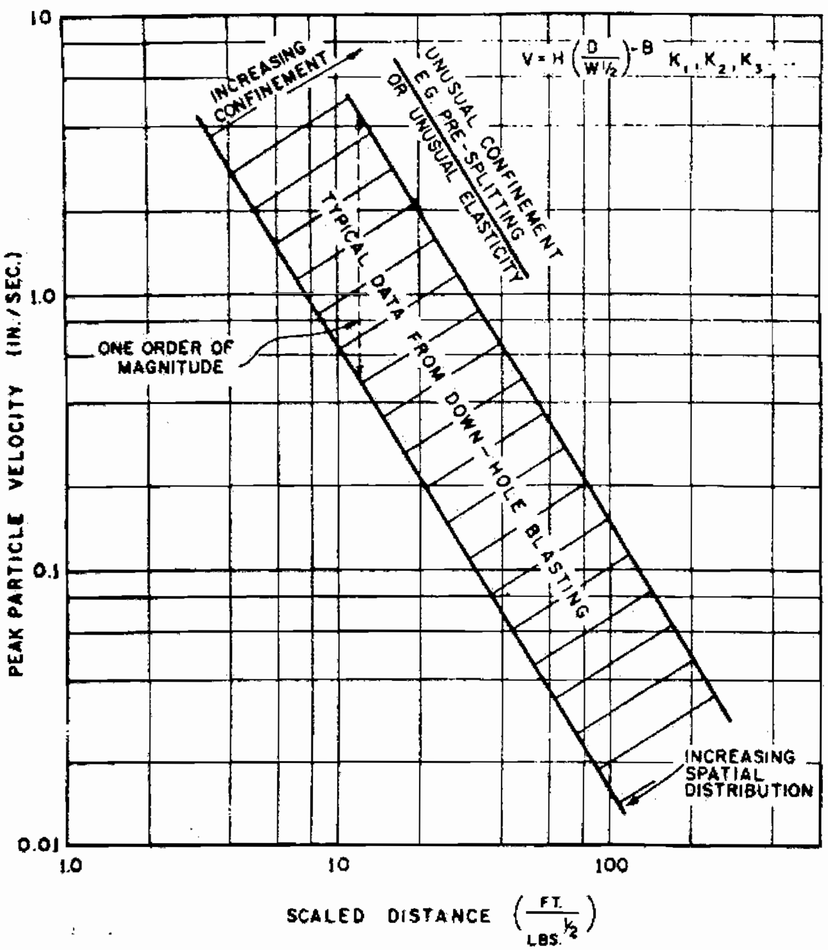


Figure 4 - Peak Particle Velocity vs Scaled Distance

attenuation initially, until low seismic strain levels are reached, at which time the attenuation should remain relatively constant in accord with the geological setting. Similarly, waves arriving at locations near the source are a complex function of several wave types, all combined. Because of different transmission velocities, these waves then separate as they move away from the source, spreading out between arrival times and generating different types of particle motions. Thus, there should be a more rapid attenuation near the source, and later an attenuation which is determined by the predominant wave, usually the Rayleigh wave. The above theories should hold for a point source. Some data show agreement with such theories, especially those taken from nuclear detonations, where it is clear that both high strain levels and a point source are correctly modeled. However, there is an interesting departure from this type of attenuation for most blasting operations. Usually, a straight extrapolation serves the purpose. Sometimes it even reverses from theory and forms a flatter slope near the source. This happens when a large number of holes detonates simultaneously. There is no such thing as zero distance in that case, and the attenuation is distorted by the departure from a point charge. Even in the case of very large blasts, such as "coyote" blasts, (a tunnel filled with explosives), the condition departs from theory by a failure to detonate instantaneously or from the center outward, - two theoretical requirements. And in the case of relatively low energy levels, there is some experience to demonstrate the validity of the same attenuation carried very close to the source. A case in point is that of small charges detonated in old concrete at Lock and Dam No. 1, Minneapolis. The writer's prediction curves were found to represent accurately the attenuation extrapolated as close as 8 inches from the source. (Tart, Oriard and Plump, 1980; Oriard, 1980). In order to measure these very unusual vibrations, it was necessary to use accelerometers having a frequency range up to 30,000 Hz and an acceleration range to 30,000 g. The writer's data cover a distance range from 8 inches to 20 miles and a charge weight range from 1 gram to nuclear devices. The curves are intended to portray that range of experience.

The broad base of experimental data mentioned above provides a convenient means of making reasonable predictions of vibration intensity. However, it is helpful for the understanding of slope stability questions to be aware of some of the factors that influence other characteristics of the vibration besides intensity, such as frequency (hence wave length) and displacement. Starting at the source, one finds an inverse relationship between charge size and frequency, for any given medium. The larger the charge, the lower the frequency and the larger the displacement. Similarly, normal wave propagation phenomena bring about a decrease in frequency with distance. The higher frequencies are more quickly attenuated. The more predominant factors relating to the attenuation of seismic waves are (1) geometric spreading, (2) selective scattering, (3) absorption, and (4) dispersion. As mentioned previously, geometric spreading is inversely proportional to the distance for body waves, and inversely proportional to the square root of the distance for surface waves. Scattering varies inversely as the fourth power of wave length (therefore directly with frequency); absorption increases with the second power of frequency, and dispersion varies with the first power of frequency.

The wave transmitting medium (geological setting) has a strong influence, also, on the frequency and displacement characteristics of a seismic wave. For example, hard massive rock will be characterized by smaller displacements and higher frequencies, whereas soil will be characterized by larger displacements and lower frequencies. Both the attenuation and the wave form characteristics are influenced by such geological factors as layering, jointing and water content, as well as the small-scale elastic properties of the medium. For example, in certain regions underlain by prominent horizontal layers of sedimentary rock, it has been noted that surface waves appear to be more prominent and persist to greater distances than is typical for regions that are more heterogeneous and/or geometrically complex.

VIBRATION EFFECTS ON SLOPES

The experience of this writer suggests that there is a frequent need for a fast, relatively simple evaluation of the stability of slopes subjected to blasting stresses. Blasting operations themselves frequenty produce slopes of sufficient height to be in need of evaluation, and they are often found in proximity to other slopes of concern, whether composed of soil or rock or some combination of materials. In the majority of cases when a question arises concerning stability, a judgment must be made rather quickly because of the financial and scheduling needs of the project which is underway. Sometimes the evaluation is required in advance of project start-up, for varying reasons, including that where there is a need to provide "documentation" that the future project will not generate public or private hazards. It happens quite frequently in such cases that there is a specific demand for a calculation using some "standard" pseudo-static method of analysis. With such methods, it is often assumed that the estimated horizontal acceleration of the predicted vibration will act as a static force in the horizontal plane in the direction of the outer slope face. Those who have performed such exercises will come to realize that they predict dire consequences in nearly every case involving blasting vibrations, despite the long history of experiences to the contrary. In spite of this lack of correlation with blasting experience, the use of such methods remains widespread.

Terzaghi's Method

The origin of pseudo-static methods of analysis of slopes and embankments subjected to vibration may have developed a very long time ago. However, the "standardization" of an approach very likely began with Terzaghi (1950),(Seed, 1979). Terzaghi described the method as follows:

"An earthquake with an acceleration equivalent n_g produces a mass force acting in a horizontal direction of intensity n_g per unit

of weight of the earth. The resultant of this mass force, $n_g W$, passes like the weight, W, through the centre of gravity O_1 of the slice abc.* It acts at a lever arm with length F and increases the moment which tends to produce a rotation of the slice abc about the axis O by $n_g FW$. Hence the earthquake reduces the factor of safety of the slope with respect to sliding from G_s, equation (1) to

$$G'_s = \frac{slR}{EW + n_g FW} \quad . . . . (2)$$

"The numerical value of n_g depends on the intensity of the earthquake. Independent estimates (Freeman, 1932) have led to the following approximate values

Severe earthquakes, Rossi-Forel scale IX $n_g = 0.1$
Violent, destructive, Rossi-Forel scale X, $n_g = 0.25$
Catastrophic $n_g = 0.5$

The earthquake of San Francisco in 1906 was violent and destructive (Rossi-Forel scale X), corresponding to $n_g = 0.25$.

"Equation (2) is based on the simplifying assumptions that the horizontal acceleration $n_g g$ acts permanently on the slope material and in one direction only. Therefore the concept it conveys of earthquake effects on slopes is very inaccurate, to say the least. Theoretically, a value of $G'_s = 1$ would mean a slide, but in reality a slope may remain stable in spite of G'_s being smaller than unity and it may fail at a value of $G'_s > 1$, depending on the character of the slope-forming material.

"The most stable materials are clays with a low degree of sensitivity, in a plastic state (Terzaghi and Peck, 1948, p. 31), dense sand either above or below the water table, and loose sand above the water table. The most sensitive materials are slightly cemented grain aggregates such as loess and submerged or partly submerged loose sand . . . . "

* Terzaghi's figure is not reproduced here because it is not needed for the purposes of this paper. Quotation was used to make sure that there were no misrepresentations of Terzaghi's statements.

In recognition of Terzaghi's eminent status in the field of soil mechanics, it is not surprising that this concept caught on and became widely applied. However, there are aspects of the question worth noting. One is that Terzaghi himself recognized the complexity of the problem, even as it related to the case of earthquakes, which is relatively simple when compared to blasting phenomena. Paradoxically, later advocates of this method often chose to use seismic coefficients which were much less conservative than those recommended by Terzaghi; yet the method normally is so dramatically over-conservative for blasting phenomena that it is usually quite misleading to use it. The reason for this apparent paradox is due to the differences between earthquake and blasting vibrations. Most blasting vibrations are characterized by relatively high frequencies compared to earthquakes. In turn, acceleration is proportional to the square of the frequency.

Thus, blasting vibrations generate relatively high accelerations for whatever particle velocity is involved (hence, strain). At the low frequencies associated with large earthquakes, an acceleration of 0.1 g is regarded as strong motion, and an acceleration of 1.0 g would be regarded as catastrophic. This is not at all true of blasting vibrations. For small charges at close distances in rock, the corresponding accelerations may be many tens of g's without necessarily being of concern. For example, the author has measured non-damaging accelerations approaching 1000 g in the walls of an operating powerhouse in Venezuela, and accelerations in the range of 20-30,000 g in the walls of an old concrete lock, as mentioned previously. It is quite clear that acceleration alone is not a diagnostic feature of the damaging potential of propagating seismic waves, or even of simple mechanical shaking. A small displacement at high frequency may have a higher acceleration than a larger displacement at low frequency, yet the latter may have more damage potential because of larger strains generated. The stress generated by a passing seismic wave is proportional to the product of the acoustic impedance of the material and the particle velocity. Therefore, there is no theoretical reason why particle acceleration should be used in evaluating strains in slopes due to passing seismic waves. Using the relationships for sinusoidal wave forms, the following relationships can be seen to illustrate the above comments:

Displacement	Frequency	Acceleration	Strain
0.001 in. (base case)	100 Hz	1 g	1 unit (base case)
0.1 in. (100 times increase)	10 Hz	1 g	10 units
10.0 in. (10,000 times increase)	1 Hz	1 g	100 units

Newmark's Method

In an effort to improve on earlier pseudo-static models of slopes subjected to seismic shaking, N. M. Newmark, in his Rankine Lecture of 1965, proposed a procedure for evaluating the potential deformations of an embankment subjected to earthquake shaking (Newmark, 1965). In this method, it is assumed that slope failure is initiated and movements begin when the inertia forces are large enough to overcome the yield resistance of the slide mass, and that movements stop when the inertia forces are reversed. The cycle may or may not be repeated. With this model, the investigator computes the acceleration at which the inertia forces become sufficiently high to cause yielding, then integrates the effective acceleration on the sliding mass in excess of the yield acceleration as a function of time to obtain velocities and displacements. The velocities are shown as functions of time for both the accelerating force and the resisting force. The maximum velocity for the accelerating force has the magnitude V given by the expression $V = Agt_o$. After the time t_o is reached, the velocity due to the accelerating force remains constant. The velocity due to the resisting force has the magnitude Ngt. At a time t_m, the two velocities are equal and the net velocity becomes zero, or the body comes to rest relative to the ground. Displacements are made in distinct, discrete steps if there is sufficient difference in the velocities of the base and the sliding mass.

For very long waves, such as those generated by typical earthquakes the slope motion can be said to be a very crude form of a mechanical shaking table, and indeed shaking tables have reproduced this type of step-by-step displacement in scale models of embankments tested in the laboratory (Seed, 1979, 1980). Such laboratory tests and actual earthquake experiences have shown that embankments can undergo substantial accumulations of discrete displacements without necessarily "failing". The Newmark method has been found to be quite useful where the yield resistance of the embankment can be reliably determined, where pore pressures do not change significantly, and where the materials do not lose more than about 15% of their original strength during the shaking (many clayey soils, some dense saturated sands and clayey sands), if the mass can initially tolerate an inertia force of the order of about 0.1 to 0.15 g without yielding, and crest accelerations are less than about 0.75 g (Seed, 1979, op sit.).

The phenomenon of accumulated displacements is normal for many rock slopes and soil slopes. For very small dynamic loads, no effect whatever may be noted. For somewhat heavier loads, small displacements may be initiated. As they accumulate, there is often ample opportunity for observation and the development of remedial measures. Most soil and rock slopes develop sufficient residual strength after the initial movements to have a controlling influence on later movements. Many slopes are not capable of undergoing sudden failure because of this residual strength. Exceptions are such cases as the first, sudden failures of rock wedges subjected for the first time to strong shaking. There is an increasing sensitivity as we proceed from the case of previously failed zones which have come to rest in new stable positions, to the case of still-moving masses, to those which have never failed but are potentially very unstable.

Unfortunately, the Newmark method does not model wave propagation phenomena. It assumes that the slope rests on a rigid base subjected to mechanical shaking. Of course, this is not at all true for blasting vibrations. There are different wave forms involved, and the wave lengths are often short compared to the slope length. Very often, there is a dramatic attenuation within the slope length of interest. There are additional complexities of a geometrical nature that are not significant in the case of earthquake shaking.

Some of the major differences which normally exist between typical earthquakes and typical blasting can be summarized as follows:

1. Boundary conditions for blasting are not usually those assumed for the slope model. Commonly, only a small portion of the

slide mass is subjected to a given motion at any given instant of time.

2. One cannot assume a single intensity of shaking, since the vibration will attenuate within the slope.

3. Not only will different parts of the slope undergo different intensities of vibration, but there are also different frequencies involved with attenuation, so that the different sections do not move in phase.

4. Different wave forms are involved, which separate with distance, so that even the duration of vibration changes, as well as the intensity and frequency.

5. The surface motion will be different from that at depth. Which motion should be considered to act on the slope? Body waves or surface waves?

6. The direction of travel and angle of incidence are important. Is the wave arriving at grazing angle of incidence to the slope surface (in which case there may be very little motion tending to stimulate sliding)? Or is it coming from an angle that may generate surface reflection of long waves? Is it realistic to consider that all possible angles of incidence need to be evaluated?

7. Considering not only the low stress levels usually involved, but the small particle displacements as well, small surface irregularities become more important in resisting slope movements.

8. Cycle duration is shorter, just as the wave lengths are shorter, thus providing less opportunity for displacements to occur.

9. The combination of small displacement and high frequency are quite significant. Mass dilation may occur without any slope displacement. The condition can be compared crudely to space tolerances in mechanical equipment subjected to vibration. If two parts are separated by a distance comparable to the particle displacement of the vibration, it is not likely that the vibration will affect them.

In consideration of the complexities of wave propagation phenomena, it is not desirable to use pseudo-static methods of analysis for the more common cases of blasting. Of course, it is possible that a history of experiences in a given geological setting, with repetitions of a given type of blasting, might permit the investigator to develop a special application of such methods by determining empirically the appropriate "artificial" seismic coefficients. However, such an approach would fall apart quickly if there were significant changes in blasting methods or site characteristics, including geometric considerations. It is not likely that such an approach would have any advantage over the simpler experience of comparing observed slope behavior to measured particle velocities.

Possible New Pseudo-Static Method of Analysis

Considering the attractive, convenient simplicity of pseudo-static methods like the Terzaghi method or Newmark method, it would seem worthwhile to pursue a similar approach that takes into account some of the physical parameters that are more characteristics of wave propagation phenomena than those considered in the former methods. Oriard and Yen (1977) presented a discussion of such an approach regarding blasting effects on unstable slopes of the Panama Canal. It was hoped that this project might offer an opportunity to gather some meaningful well-controlled field data during the proposed deepening program (Oriard, 1980A). However, the effort was cut back due to political changes. Consequently, the writer has not yet had the opportunity to gather any field data which could be used to evaluate the suitability of such a method. A considerable effort would be required to develop such a method and to check its validity against known performance of identified slopes.

As a beginning approach to such a method, the following concepts could be pursued initially, - perhaps modified later as needed.

F_{sta} is the static shearing force, primarily the downhill component of the gravitational force. It may include pore water pressure, if any exists. For a generic èlement in the slope, the static driving force could be expressed as

$$F_{sta} = (\rho\ g\ h \sin\beta) (\cos\beta\ dL)$$

where ρ = mass density

g = acceleration of gravity

h = vertical dimension of slope element

dL = element length along slope angle

β = slope angle

F_{dyn} is the dynamic load induced by blasting. The dynamic force is transient, cyclic and varies in direction and magnitude. There is no known closed-form solution for F_{dyn}. The dynamic force depends on the many variables previously mentioned. Although the degree of conservatism is not known for actual field conditions, perhaps a reasonable, though conservative, approximation could be represented by

$$F_{dyn} = (\rho\ c_s\ v) h$$

where c_s = shear wave velocity

v = peak particle velocity

F_{res} is the residual force that may exist in the rock slope, such as that which may be due to tectonic stresses, chemical stresses, and the like. For a simplified analysis, F_{res} may generally be neglected. For failure,

$$F_{sta} + F_{dyn} + F_{res} \geq \text{Resisting Force}$$

A further reasonable assumption for slope failure would be a requirement for the slope particles to move in unison, that is, an in-phase velocity field, thus limiting the zone of interest to one which has a dimension less than 1/2 wave length,

$$L \leq \lambda/2$$

For the purposes of an initial evaluation of this method, it could be assumed that the rock strength under combined static and dynamic stress should be less than its peak static strength. That is, we could make a beginning assumption that

$$F.S._{dyn} = \frac{R}{F_{sta} + F_{dyn} + (F_{res} = 0)}$$

where R is the peak static shearing resistance

$$R = (\rho\ g\ h \cos^2\beta)\ dL \cdot \tan(\phi_r + i)$$

where ϕ_r = friction angle

i = equivalent friction angle increase to account for such factors as joint roughness.

Pursuing this concept, one can prepare families of curves such as those in Figure 5.

Further research is needed to determine how well field experience will agree with the predictions. As in all other aspects of explosives engineering, it is anticipated that there will be a need for judgmental factors or "coefficients" to relate the calculations to experience. Depending on the range of such needed coefficients, the practicability of the method may then be assessed. Although there are many obvious theoretical shortcomings to the method, there seems to be a chance for somewhat better correlation than with previously used pseudo-static methods. And it seems more likely that there would be a possibility of better correlation with shallow slope failures than with deeper ones. For deeper failures, the boundary conditions would surely become increasingly more important. In all cases, it is very important to study the slope responses to physical stimuli and to determine the time history of repeated responses. If the measured displacements are decelerating the failure is regressive and there may be no need for further action at that time. If, however, the displacements are accelerating, the failure is progressive, and action may be needed quickly, such as a change in blasting methods, or remedial work on the slope, such as suggested in Figure 6 which represent previous actual field experience. (Oriard, 1971)

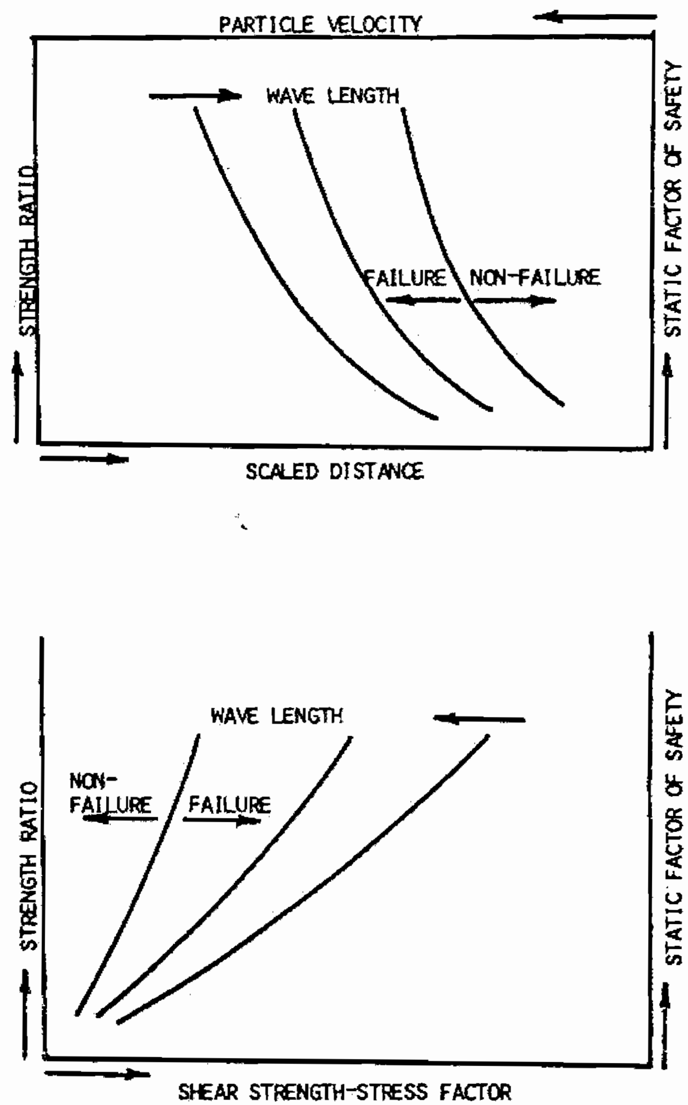
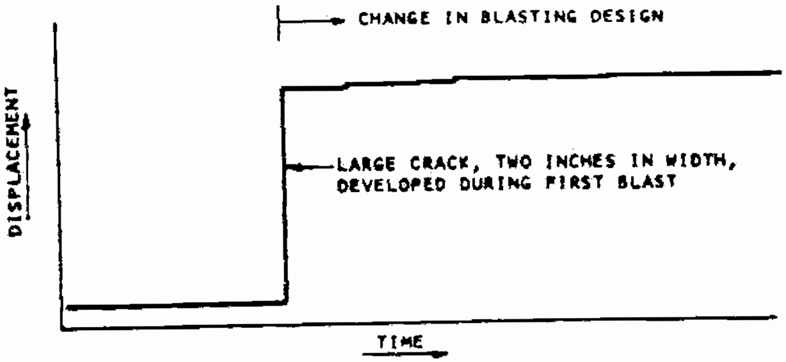


Figure 5 - Possible Slope Failure Criteria



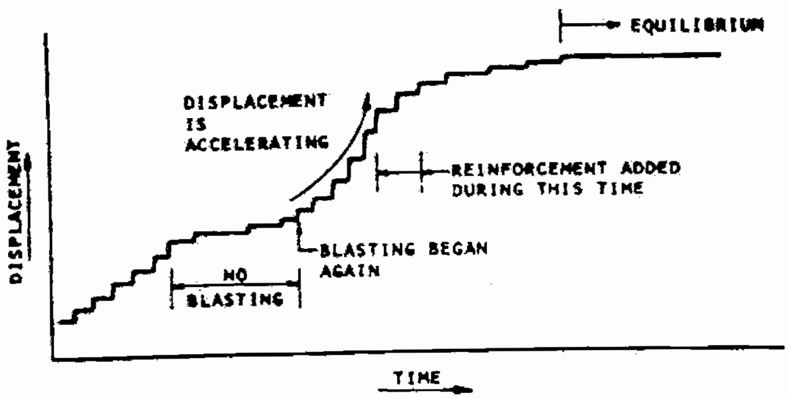


Figure 6 - Illustration of Progressive and Regressive Dynamic Responses to Blasting Vibrations.

Dynamic Methods of Analysis

Dynamic methods of analysis do exist, and have shown significant improvements in recent years. With the increasing sophistication of computerized analytical techniques now available, such as the finite element method and the finite difference method, it is now possible to analyze dynamically the simpler vibration models and most types of slope models. At least two types of vibration input can now be analyzed quite accurately. One is that of a rigid-base model subjected to any vibration history of interest. The other is that of a simple form of stress wave propagating through a continuum. Unfortunately, these methods are relatively time-consuming and expensive, and still face very formidable problems when dealing with blasting phenomena. Wave propagation models become extremely complicated, and the range of possible cases becomes discouragingly large. And, of course, the slope must be accurately modeled if the methods are to have reasonable validity.

On many projects, there is neither the time nor the financial resources available for the development of an acceptably accurate dynamic model. In such cases, judgements must be made without the benefit of such analysis. Two of the most common approaches to such cases are (1) program the blasting in order to limit stresses to conservative values, and/or (2) monitor slope behavior carefully to observe the first signs of any adverse reaction.

Blasting is Controllable

One very important distinction between blasting and earthquakes, in addition to those mentioned previously, is that blasting can be controlled by design. In most cases, it will be found possible to limit blasting stresses to acceptable levels without adding any significant financial burden to the project. If, however, a more precarious condition exists, it is normally a simple matter to begin blasting on a limited scale and build up to a larger scale on a programmed basis while monitoring slope behavior. Of course, this not true of earthquakes. The controllable aspects of blasting, whether the vibration intensities are controlled by distance or by design parameters, permit a close scrutiny and detailed instrumental monitoring of the slope at low vibration intensities, and at increasing vibration intensities, in accord with the wishes of the investigator. It is relatively rare that the question must be answered for the case of a single, large event, with no opportunity for preliminary observation, although the latter case sometimes arises (Oriard and Jordan, 1980).

Dynamic Stability

The dynamic stability of a slope is very closely related to its static stability. Those same physical properties (especially in-situ larger-scale mass characteristics, and properties of weak planes) that render a slope unstable under static loading conditions contribute to its lack of stability under dynamic loading. That is, the higher the static factor of safety, the higher the dynamic factor of safety, in general. However, some of these factors, such as surface irregularities on rock slopes, have varying relationships to stability in accord with the type of dynamic loading that occurs, not merely its numerical value of acceleration or velocity, as previously discussed.

It is anticipated that the state-of-the-art regarding static stability will be discussed at length in this conference. For the purposes of this paper on blasting effects, only a few general comments will be made regarding static stability, in order to complete the discussion of blasting.

Three of the most important factors relating to stability of rock slopes are (1) size, location and orientation of critical discontinuities, (2) the shear strength along these discontinuities, and (3) the pore pressures on these discontinuities. A person wishing to evaluate dynamic stability would proceed initially in the same manner as an investigator evaluating static stability. He would investigate such factors as

1. Geologic history, including weathering processes and profiles, geologic age, rates of steepening or flattening of slopes through natural processes and/or the activity of man.

2. Stress history and anticipated in-situ stresses.

3. Climatic and hydrologic history, past and present.

4. All factors relating to the present "mechanical" conditions, such as type of materials, bedding and jointing (frequency, orientation, fillings, openness, irregularity, etc. (see Goodman, 1981; Barton, 1981, and others).

5. Any previous dynamic history, such as earthquake activity, previous blasting activity, or steady-state vibration sources.

CASE HISTORY - PRECARIOUS SOIL SLOPES

The following case history will illustrate some of the problems that are often encountered when dealing with blasting effects and slope stability, and one of the approaches to dealing with such problems. The case involves certain unstable tailing dams. The slurry formed by the fines left over from ore milling, mixed with waste water, was pumped by pipeline to waste areas. As the slurry began drying at the perimeter, the dry, fine sand around the perimeter was reworked with bulldozers to form dams to contain additional slurry. The process was continued, simultaneously building up the dams and filling more tailings behind them. One of the dams had a crest height of approximately 43 m at the time of this investigation. Two or three local slope failures had occurred, and an investigation led to the conclusion that the dams were statically precarious and incapable of

withstanding the shaking action of a moderate earthquake that might occur in the region at some time in the future. Consequently, a decision was made to place a rockfill buttress against the steeper, lower portion of each embankment (Figure 7). A suitable quarry sit was found nearby, from which the rockfill material would be obtained The rockfill was to be high-quality material, requiring blasting. Thus, a large-scale quarrying operation would be required in the vicinity of statically precarious tailings embankments. A total volum of about 1,300,000 tons of rockfill would be required. In addition, there was a need to blast drainage trenches immediately in front of the toes of the embankments. Thus, one of the interesting aspects o this case was the need to consider three different kinds of vibratio (1) low frequency vibration generated by an earthquake, (2) mid-frequency vibration generated by quarry blasting, and (3) high-frequenc vibration generated by trench blasting.

One would consider the factor of safety of existing conditions to be about 1.0 $\pm$. Theoretically, the embankments were incapable of tolerating any vibration. Even after the construction of the buttre ses, they would be capable, theoretically, of withstanding an acceleration of only 0.12 g (the design earthquake). If the usual pseudo static models were considered valid, no remedial work could be done because the blasting would generate unacceptable vibrations. Fortunately, experience has demonstrated that certain vibrations can be tolerated under such circumstances, and that the higher the frequenc of the vibration, the greater the acceleration that can be tolerated (for equivalent strain).

The following table illustrates the particle motion parameters of interest, showing the range in particle velocities and displacements for various frequencies, assuming a constant acceleration of 0.12 g.

Acceleration	Frequency	Velocity	Displacement
0.12 g	0.1 Hz	75 ips	120 in
0.12 g	1.0 Hz	7.5 ips	1.2 in
0.12 g	10 Hz	0.75 ips	0.012 in
0.12 g	100 Hz	0.075 ips	0.00012 in
0.12 g	1000 Hz	0.0075 ips	0.0000012 in

The reader can see from the above figures that if we are given a constant acceleration as a limit, we then find that velocity is inversely proportional to the first power of the frequency, and that displacement is inversely proportional to the square of the frequenc

At limiting equilibrium, if the acceleration is limited to zero, due to instability, no vibration whatever can be tolerated. Hence, in theory, no blasting can be tolerated. Even if we assume that the blasting takes place after the completion of the work, at which time

Figure 7 - A rockfill buttress being placed on the lower slopes of a precarious tailings embankment. Blasting was required at the toe of the slope and in a nearby quarry.

the embankments can theoretically tolerate an acceleration of 0.12 there is still a serious limitation. There is no possible way tha any trench blasting could be done at the toes of the embankments without greatly exceeding 0.12 g. If we were to believe that accel eration is a valid criterion, the exercise is self-defeating, beca there is a rapid increase in frequency with reduction in charge si Even though particle velocities and displacements might be reduced reducing charge size at close distances, we might begin to pick up high frequencies that do not exist at greater distances. Charges c only several ounces of high explosives may generate accelerations c the order of 10,000 g to 30,000 g within the first several feet, as mentioned previously (Oriard, 1980). If we seriously believed in acceleration criteria, we would want to make every blast act like a small earthquake, and that would not be a wise approach.

The writer recommended using the observational approach in this case, believing that any of the "standard" methods of analysis wou only be misleading. It was obvious that high accelerations would b generated by the trench blasting, and that moderate levels of accel eration would be generated by the quarry blasting. If neither mod- erate nor high levels of acceleration could be tolerated, nothing would be gained by making the calculations.

With the observational approach actually applied to this case, t latest techniques in blasting technology were used to control the v brational particle velocities to levels considered to be conservati and would still permit the work to proceed at a large scale. There was no measurable sacrifice due to blasting controls, beyond the fi few days of initial trials while the embankment behavior was being very carefully monitored. The embankments were monitored for dis- placements, changing pore water pressures and phreatic water levels If there were no significant increases in pore pressures, and the e bankments did not undergo any displacement, no change in stability would take place. It was concluded that a series of smaller displa ments would occur and accumulate before there would be any danger c a significant failure. In the case of the trench blasting, at high stress levels, any single displacement could conceivably be greater but would be limited to a small portion of the embankment immediate adjacent to the blasting area. Thus, even though the mechanisms could be different for the two types of blasting, there would still be an incremental development of any significant displacements.

Two types of piezometers were used. One type was the isolated-t type, consisting of a porous tip installed at the specific point of interest. These are more sensitive and react more quickly than the other type which was used, - the open-well piezometer. With the fi type, pore pressure changes could be monitored within a few minutes after a blast. The open-well piezometers provided information on changes in the over-all phreatic line in an embankment. Both assis in the assessment of stability, or change in stability.

All blasting vibrations were monitored. Seismographs were placed in suitable locations to record bedrock vibrations and embankment responses. The embankment responses often showed an amplification of the order of 4 times greater than that of the bedrock base. Of course, for the trench blasting, there was a significant attenuation from the toe to the crest, although the same relative amplification could be detected between the tailings and rock.

Quarry blasting began cautiously and increased in scale while the embankments were monitored. Initially, there were very minor increases in pore pressures, but these were quickly dissipated (typically, in 15 minutes to several hours) with no observable longer-term effects of concern. Very close observations were continued until the quarry blasting had been increased progressively to a point beyond the level desired for long-term, continued operation, then reduced to that for the long-term program. The time intervals between embankment observations were then slowly increased as no disturbances were noted.

Figure 7 illustrates the field setting. Figures 8, 9 and 10 illustrate typical bench-mark readings, piezometer locations and piezometer readings.

Despite the obvious precariousness of the embankments, there were no adverse effects of any type observed during the six-month period of blasting, involving well over 200 blasts.

Vibrations were monitored with velocity gages rather than with accelerometers. Accelerations were neither measured nor calculated, because they were not regarded as being diagnostic of either shaking intensity or damage potential. However, quarry blasting designs were programmed in such a way as to keep the predicted range of accelerations at or below the 0.12 g design earthquake acceleration (not the theoretical zero acceleration assumed as a limit before the buttreses were placed). The trench blasting probably generated accelerations of the order of 50 g near the toe, of the order of 1.0 to 1.5 g 15 m within the embankments and about 0.5 g at 30 m (meters).

In this case, the observational approach was selected over analyses that were regarded as inappropriate. At the same time, however, the writer would like to repeat the opposite concern about being too liberal with such analyses when applied to earthquakes generating very low frequency ground shaking, recalling the recommendations of Terzaghi (1950) and Seed (1979) mentioned previously. The emphasis should be placed on recognizing the differences in these different types of vibrations, and in treating them accordingly.

The experience of observing very high accelerations (even high velocities and high strains) that are not damaging to slopes thought to be statically precarious raises some interesting questions. Some of these questions cannot be answered merely on the basis of short wave

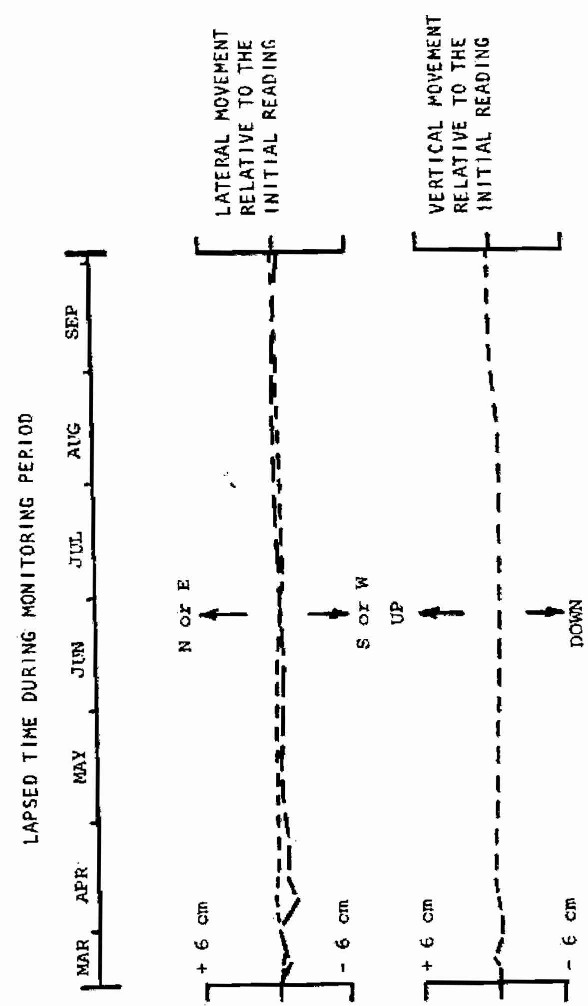


Figure 8 - Typical Slope Displacement Monitoring Data

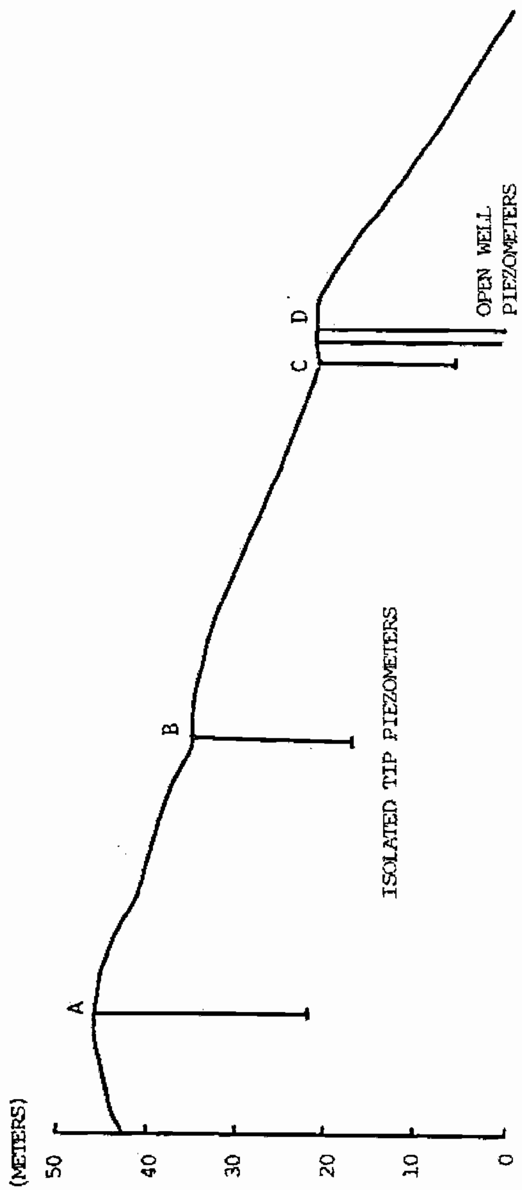


Figure 9 - Typical Tailings Embankment Section with Piezometers

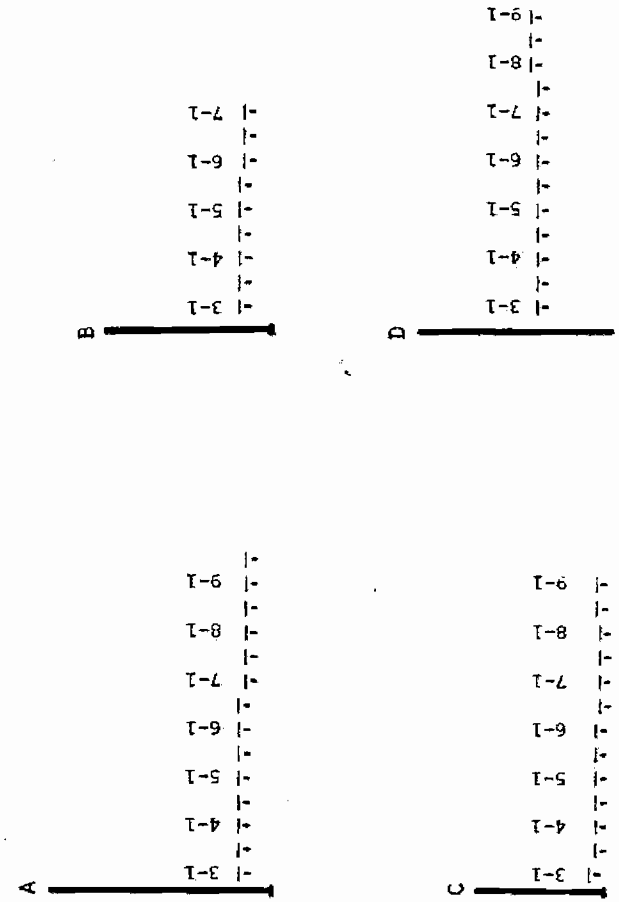


Figure 10 - Typical Long-Term Monitoring Data from Piezometers

lengths alone, although that is a factor of great importance. That would lead only to the conclusion that the damaged zone should be of limited dimension. Of course, one would expect the damage to occur where the stresses were highest, - near the blasting. This type of experience suggests that we need to look very carefully at boundary conditions. For example, a failure might be possible if a small zone in the embankment could be artificially bounded by failure planes, but that it must be greatly strengthened by being bounded by a continuation of the same material well beyond the distance at which high stress levels would be found. That is, the potential zone of failure is supported or held by adjacent material not under the same level of stress. Thus, the true boundary conditions are not necessarily the bounds of some potential mass of sliding, but may be determined more properly by understanding the character of the vibration.

CASE HISTORY - PRECARIOUS ROCK SLOPE

The following history was selected to illustrate a situation completely different from that of the previous case. In the following case, steep rock slopes were subjected to the direct rupturing and tearing actions of blasting operations, leaving damaged rock layers in an unpredictably precarious situation.

The case involved a quarrying operation in steeply dipping layers of limestone. Layers were typically from 20 to 30 meters in thickness, separated by prominent discontinuities with very little shear strength. Blast holes were drilled vertically by drills which were lowered down the slopes by ropes and cables. Drilling and blasting began at the toe of a layer, breaking off sections which then tumbled by gravity along the surface of the next underlying layer. The broken rock was picked up at the toe of the slope and hauled away. Successive blasts continued up the slope to higher and higher elevations. The operation is illustrated in Figure 11.

The experienced reader will recognize this procedure as being the same as that which is used to bring down and dispose of precarious rock wedges or other potential rock slide zones, with the difference in the latter case that a larger portion of the precarious zone is usually blasted to ensure the failure of the entire zone.

In the present case, a slope failure occurred at the time that a drilling crew was working on the slope, killing and burying the men. It is not known, in hindsight, all the factors that contributed to the failure, and in what proportion. There is no doubt that the previous primary blast caused damage in tearing away from the rest of the layer. It is not known whether or not compressed air from the drilling operations might have been injected inadvertently into the parting between the layers, nor how many drills were actually in operation at the moment of the failure. It is not known if there were any visual indications of slope loosening or displacement prior to the failure. Nevertheless, it is hoped that the reader will recognize the hazards in

Figure 11 - Precarious rock layers in a limestone quarry. The massive limestone layers are unstable only when undercut by the quarrying operations. Otherwise, they remain stable (see text).

this type of operation, and take steps to avoid them.

The writer was asked to recommend a method for removing the remainder of the unstable, undercut layer, and to develop a new, safe quarrying plan. Briefly, the safe removal of the undercut area was accomplished by placing the drills to the side of the layer and drilling horizontal holes into the unstable rock, keeping men and equipment off it. With the proper design of hole length and sequence of detonation, it was possible to fragment the rock for product use, and avoid bringing the layer down merely as a slope failure (which would have required very expensive secondary blasting below). Future quarry development called for benching from the top down, although gravity could still be used in place of hauling units, merely by pushing the muck to the steeply dipping bedding planes and letting it slide to the bottom as before.

Because of the possibility of a sudden slope failure at any time, the writer did not consider it safe to use the approach described previously for the tailings embankments. Neither was there any attempt whatever to perform any type of analysis. In the judgment of the writer, it would not have been possible to determine the stability with sufficient accuracy to be meaningful, even though the slide surface was unusually accessible for examination and/or testing. The remaining rock might have failed at any time due to simple gravity loading, or it might have withstood the remainder of any blasting activities performed according to the previous quarrying methods, since there had been a very long history of use of the previous methods before this accident.

It was this writer's opinion that it would not have been wise to attempt to draw a very fine prediction line between failure and no failure in this case. On the other hand, had it been necessary to continue placing men and equipment on the precarious slope, it would have been possible to develop an observational approach different from that described for the tailings embankments. Water was not present in this slope, so pore pressures were not of interest. Displacements could not have been permitted to accumulate to the same degree as those in a soil embankment. Therefore, the monitoring would have to be tailored to a much greater degree of sensitivity to early warnings of displacement. For example, acoustic emissions and very sensitive displacement monitors (such as LVDT's) could have been used. The preferred approach, where personnel safety was so important, was to avoid the problem altogether.

Repeating for emphasis, it is this writer's opinion that methods of evaluation, methods of observation, and methods of blasting must be highly site specific.

SUMMARY

The existing state of the art regarding blasting effects on slopes relies heavily on the experience and judgment of individual specialists. It might be called a technical art. It is not an exact science at all.

The physics of explosions is quite well understood, as is the rock breakage process. It is not expected that additional research will bring about any changes in fundamental concepts, but only in refinements in applications.

With the wide range of explosives products and methods now known, it is possible technically to exercise any degree of precision that may be desired in the rock blasting and excavation process. The limitation on this activity is not that of technology, but of cost. Excessive costs, of course, may prohibit the use of certain methods on a particular project, or render a particular project impracticable. Unfortunately, the success of blasting techniques depends very greatly on the skill of the individual blaster, primarily on his ability to judge the many details of the site that influence the results, and how to adapt blasting technology to best suit those specific site conditions. In addition, virtually all sites are somewhat variable, and require adaptations as the work progresses. Thus, there are certain aspects of blasting which will remain a technical art for the foreseeable future. The limiting factor is the inability to determine in advance all significant details of the site.

The physics of single seismic waves is quite well known theoretically. For any given single wave type, in any given single material, computational procedures exist for developing synthetic seismograms which can be considered fairly representative. However, there are so many possible combinations of wave forms and particle motions for a blast detonated in even relatively simple field conditions that the problem becomes very complex in most cases.

Knowing the particle motion at a particular point is not the same as knowing how to model the complex motion within a much larger zone. There are different wave forms with different velocities and different particle motions (different wave lengths, frequencies and displacements), attenuating at different rates, with different laws relating to their transmission, refraction and reflection. This complexity is compounded by very important geometric relationships, such as angles of incidence to the zones of interest. Wave lengths are very important because they limit any particular phase of motion to a zone which may be significantly smaller than a potential slide mass. In that case, the boundary conditions are not those determined by the slide planes of the slide mass, but by the dimensions of the traveling seismic waves. In many cases, it is not acceptable to assume that the particle motion is that of a rigid-base model.

For very long waves, such as those generated by earthquakes, it is possible to consider a slope to be a small model with a rigid base. Such a model lends itself to pseudo-static methods of analysis. Both Terzaghi's method and Newmark's method have been used successfully in such cases, when properly combined with experienced judgment. However, these methods are inappropriate for many blasting cases, and will often lead to very misleading conclusions, being increasingly conservative as the vibration frequency increases.

It is possible that a new pseudo-static method of analysis could be developed which would have a better chance of correlating with actual field experience relating to blasting phenomena. Such a method would have to consider some of the physical parameters which are more closely related to wave propagation, such as wave length and the strain induced by a passing wave. This paper suggests a beginning approach to such a method. Shortcomings of the method are recognized and it is expected that it will always be necessary to exercise a considerable amount of field judgment in the application of any such method.

Dynamic analytical procedures have been developed, and have been applied successfully to earthquake analysis. The most common method employed for such analysis is the finite element method, although finite difference methods are often employed for wave propagation phenomena. Future refinements of these methods may bring about suitable techniques for blasting analysis, although the complexities are formidable, and there is continuing doubt about our abilities to model a slope with sufficient accuracy to justify the time and expense for the use of such methods as applied to blasting.

As with many other aspects of the over-all question, the most serious limitation appears to be our inability to determine with sufficient precision all of the significant properties of a slope in advance of observing some aspect of its behavior. This limitation seriously impairs the development of any analytical procedure which would be suitable on a routine basis for blasting operations. At the present time, it is common to rely on the judgment of experienced specialists and to monitor slope behavior.

Fortunately, the vast majority of cases permits an observational approach which satisfies most project needs. One of the reasons is that blasting vibrations can be controlled technically to any level of interest. This permits the vibrations to begin at any level desired and to increase to any level desired, while one observes the slope behavior to the desired degree of precision. The monitoring procedures and degree of precision required are highly site specific. Items of common interest may include displacement, pore pressure, strain and acoustic emission. If the rate of response of the observed parameter is seen to accelerate, or to occur at an unacceptable level, remedial action of some sort is suggested.

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Question

Can seismic refraction profiling provide parameters for static and/or dynamic slope stability modeling. How.

Answer

Yes, conventional seismic refraction profiling techniques can provide useful information relative to such factors as the weathering profile, such as the progression from soil through decomposed rock to fresh rock, for example, and thus assist in the identification of potential slide planes. Other forms of layering can be identified and quantified, also, as long as there is an increase in velocity with depth. A series of such profiles will reveal lateral variations in material properties as well. And, of course, other techniques can provide additional information. For Example, cross-hole techniques are commonly used to measure shear-wave velocities to calculate in-situ dynamic moduli, parameters that are commonly used in both static and dynamic analyses.

Some persons have used the ratio of P-wave velocities (laboratory versus field values) to evaluate the character and frequency of jointing, for example.

On the other hand, it can be very misleading to attempt a direct correlation between wave velocities and stability unless one were to include other important relationships. The two case histories in my paper are good examples of this fact. There would have been only a single P-wave velocity for each of the two cases, - a uniform, very high velocity for the hard rock site, and a uniform, very low velocity for the tailings embankment (except for a change at the water table). Velocities, per se, would not have been useful information in either of these cases.

Question

Although presplitting does not entirely retard transmission of vibration, it does allow preservation of the rock strength. Could you

comment on this benefit in relation to open pit stability.

Answer

Some form of cautious perimeter blasting, including presplitting is indeed beneficial in preserving the integrity of final bench surfaces. For this reason, these methods have become widely used in open pit mining. In the majority of cases, I prefer cushion blasting or smooth blasting in preference to presplitting for the reason that it is more economical and offers more freedom in the drilling and blasting of the next row of holes.

There is a distinction between the effect that these methods have on preserving the integrity of bench faces and the question of benefit to the stability of the large-scale average slope behind the bench faces. Pre-splitting should not be relied upon to serve as any type of isolation device or barrier to the transmission of significant vibrations to the pit slopes.

Question

With large scale blasting in open pits is there a possibility of low frequency vibration, more characteristic of earthquakes, causing failure at pit walls distant from the shot.

Answer

Your concern is well founded theoretically. Fortunately, we are assisted by Nature in this question, however. The lower frequencies become more prominent at greater distances where the intensity of motion is lower. At most sites, the low-frequency surface waves are not well developed in the source vicinity, though they may become very pronounced at greater distances. This wave development is a function of both site geology and blasting design. Of course, we have no control over geology. We can expect more pronounced surface waves (lower frequency) in well defined soft-rock layers overlain by deep soil cover. Higher frequencies will be found at a hard-rock site. Two of the controls we should watch in blast design are (1) the maximum size of any single charge, or group of adjacent charges detonating simultaneously, and (2) the velocity with which the detonation sequence passes along a bench surface (so that we do not unwittingly provide constructive reinforcement of Rayleigh waves or flexural waves.

Question

When is a delay interval an effective delay interval for minimizing (a) overbreak, and (b) ground vibration.

Answer

Popular wisdom has it that the minimum delay interval should be 8 milliseconds or more. This delay interval is often specified as the minimum effective delay for vibration control in civil construction projects, and appears similarly in the OSM regulations. However, there is no sound technical basis for that specific number since, among other things, it does not consider other factors which are at least as important, such as the distance between consecutively firing charges, the size of the charges, or the elastic properties of the rock. The concern develops from the theory that two sine waves are partially additive if the second arrives during the first quarter cycle of motion generated by the first wave; - therefore, longer delay is better. However, the exact number is meaningless without other considerations, since reinforcement can also occur theoretically at any other whole-number multiple of period intervals and/or any distance interval that corresponds with the wave velocity through the rock. For example, 25 milliseconds for holes 25 ft. apart has the same relationship to constructive wave reinforcement as 5 milliseconds for holes 5 ft. apart. To avoid this "apparent" dilemna, Langefors advises readers to wait for several oscillations of the significant energy to die out. Unfortunately, either recommendation (8 ms, or several oscillations) may become impractical or even pose serious difficulties or hazards on some projects. Fortunately, the concern is usually unwarranted. Constructive wave reinforcement is rare. It is even more rare for the reinforcement to be sufficient to be of concern. For further discussion of the theory and a review of field data on this subject, you may wish to read Oriard and Emmert, 1980.

Question

In view of the sensitive relationship between permeability and joint width and the opening of joints by the action of blasting, could you give any evidence for a local decrease in stability due to blasting (bench scale) but an over-all increase in stability (large scale) due to drawdown resulting from increased permeability.

Answer

No, I have no such evidence and would doubt that it exists for the large scale. The action of blasting in the opening of joints is restricted to a zone in very close proximity to the blasting. It does not extend into the slope a sufficient distance to affect large-scale drawdown, only that near the bench faces.

Question

It would seem feasible, using limit equilibrium methods with slices to attach a different vector acceleration to each slice and thus crudely model high-frequency, short-wave-length, high-acceleration, blast-induced vibrations. Has this been done. By whom. How.

Answer

I am not aware of this approach being used before, but that would not necessarily mean that it hasn't been done. Although such an approach

seems attractive intuitively, it is my opinion that the judgements that would have to accompany such an analysis are as determining as the analysis itself, -just as Terzaghi's method will give the right answer if the analyst knows what seismic coefficients to use. Your suggestion is a step in the right direction because it would have the net effect of using a lower "effective" value of acceleration. But, how does one handle the boundary conditions, or determine them in the field? If the adjacent zone to the one under consideration is of equal mass and 180 degrees out of phase, the net effect is zero, assuming that the wave motion and gravity act in the same direction. This result may also be non-representative. Source location and direction of wave travel then become critical. We come back to the same dilemma: Which approach is more acceptable? To multiply the wrong answer by a judgment factor to get the right answer? Or merely to form an estimate of the right answer without the benefit of the wrong calculation? This dilemma forces us into the same operating mode that is common in static analysis, - that of observing the first sign of physical response to a known force. Of course, we must use a monitoring system that is appropriately sensitive for the site in question.

Question

Would you consider the effects of vibrational acceleration on the abrupt pore water pressure build-ups or increases which could decrease the shear strength for stability analysis. If yes, how would you introduce this concept into stability analysis.

Answer

In my opinion, one should always be concerned about the presence of water in a slope or embankment. Interestingly, experience demonstrates that the duration of shaking may be at least as important as the intensity of the shaking. With soils, it has been demonstrated many times that the exact number of oscillations at a given strain level is a critical factor in determining whether or not a failure will occur. A few oscillations less and no damage occurs; a few more and there is a disaster. One of the most dramatic examples was the terrace failure near Anchorage, Alaska, during the Good Friday earthquake of 1964. An interesting contrast is that a densification rather than failure may occur when a saturated embankment is subjected to a transient, high-frequency vibration of short duration. The response in rock may be somewhat different, depending on individual block size, the prominence of jointing, etc. We know of th potentially damaging effects of abrupt pressure increases from explosive gases, hydraulic shock or compressed air, causing local block motion. We know also of the increase in hydraulic head due to the dilatancy generated by high strain levels from earthquakes. For an open pit mine slope, I would recommend the approach described in my paper for the tailings dams, but to a greater degree of sensitivity, - monitoring the response to pre-programmed blasting loads.