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SOME CASES OF RESIDUAL STRESS EFFECTS IN ENGINEERING WORK

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FUELS AND MINING PRACTICE DIVISION

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SOME CASES OF RESIDUAL STRESS EFFECTS IN ENGINEERING WORK

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Introduction

AS STATED in its title, this conference is on the state of stress in the earth's crust. Information has been presented on fundamental concepts, methods of measurement, and principles of design. In addition, the chairman of this session requested that some actual cases be prepared that describe the effect on engineering projects from detrimental fracture and flow that are a result of residual stresses. The following cases occurred in Canada on both mining and civil engineering projects.

Floor Upheaval in an Open Pit

In the development of an open pit in Ontario it was necessary to strip up to 160 ft of overlying rock before the ore bearing formations were reached. Under about 10 to 15 ft of glacial till there was between 95 and 160 ft of Paleozoic limestone of the Ordovician Period in the Black River group of formations. Figure 1 is a photograph of the full depth of this limestone overlying the pre-Cambrian sediments and intrusives.

The basal deposits in the Black River formations consist of shales, some sandstones, dolomites, and limestone. Although the thickness of the basal formation varies, its order of magnitude is 100 ft. Above these basal deposits there is a middle phase in the group comprising pure, thick bedded, crystalline limestone about 150 ft thick. The average

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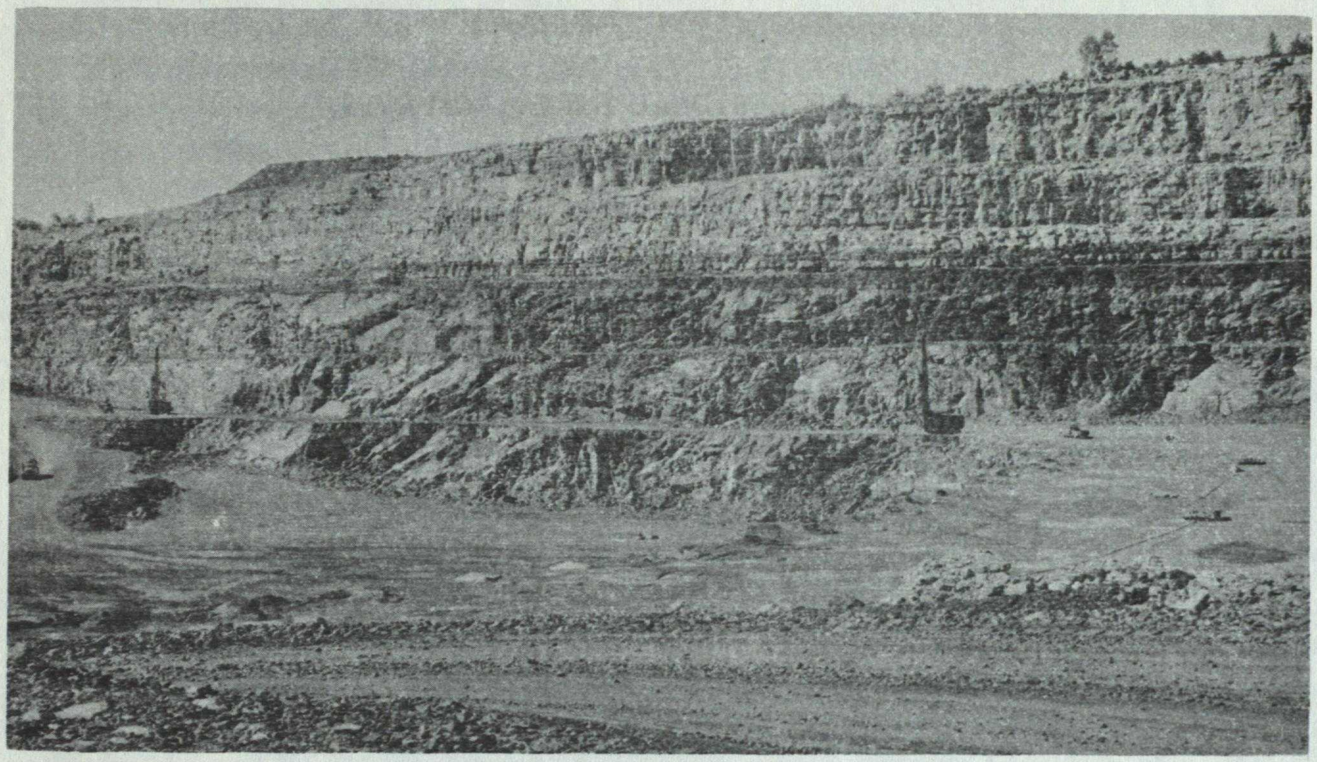

Fig. 1—Top of open-pit wall
in Paleozoic limestone.

compressive strength of the core samples obtained in the limestone at the open pit was found to be 16,900 psi, with a modulus of deformation of 6.1×10^6 psi.

The pre-Cambrian rock containing the ore consists of intensively intruded sediments. In general the rocks are hard and fairly massive, the actual types being granitic-gneiss, syenite, diorite, gabbro, limestone, and others.

During the stripping of the Paleozoic limestone the pit had been developed to the approximate dimensions of 1000 x 2000 ft, with a depth of 50 ft (see Fig. 2). Quite suddenly, in the month of January, the bottom of the pit cracked and heaved a maximum of about 8 ft within a few minutes. Figure 2 shows the location of the initial crack, which in time propagated itself.

Figure 3 shows the initial crack shortly after it occurred. It can be seen that about 50 ft of ground on either side of the crack had been deflected upwards. At this location the minimum thickness under

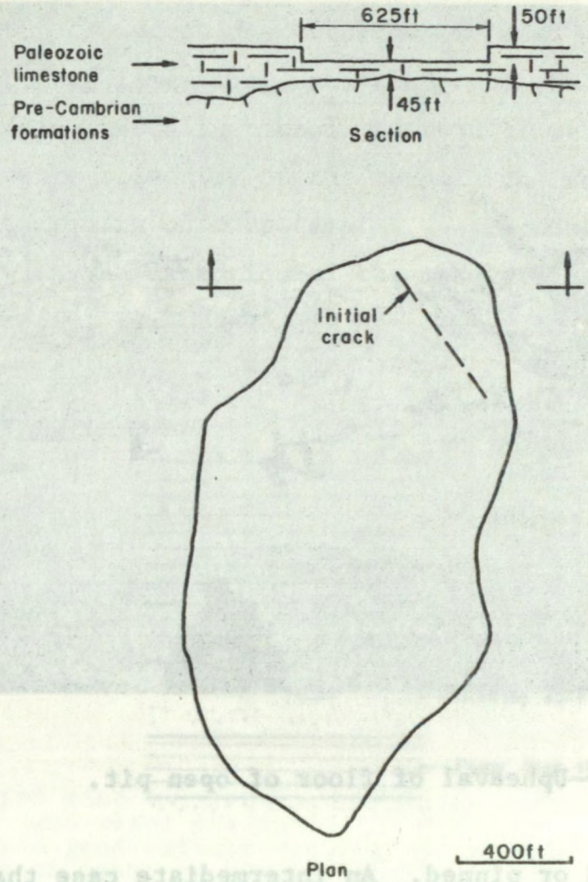

Fig. 2—Plan and section of open pit at time of floor upheaval.

the pit of the Paleozoic limestone over the pre-Cambrian rock was 45 ft, with an average thickness of about 55 ft.

If it is assumed that the upheaval was a case of elastic instability, the stress required to cause the cracking can be calculated. The geometry is rather awkward; however, the average width of excavation perpendicular to the crack can be used. This dimension would provide a member 1200 ft long.

The Euler formula for determining the critical stress for buckling has been used. The modulus of deformation of 6.1×10^6 psi is obtained from the tests on the core samples. The effective depth, visualizing a reduced moment of inertia due to natural joints in the rock, is assumed to be half the minimum thickness, or 22.5 ft. The end conditions of the formation as they pass under the walls can not be

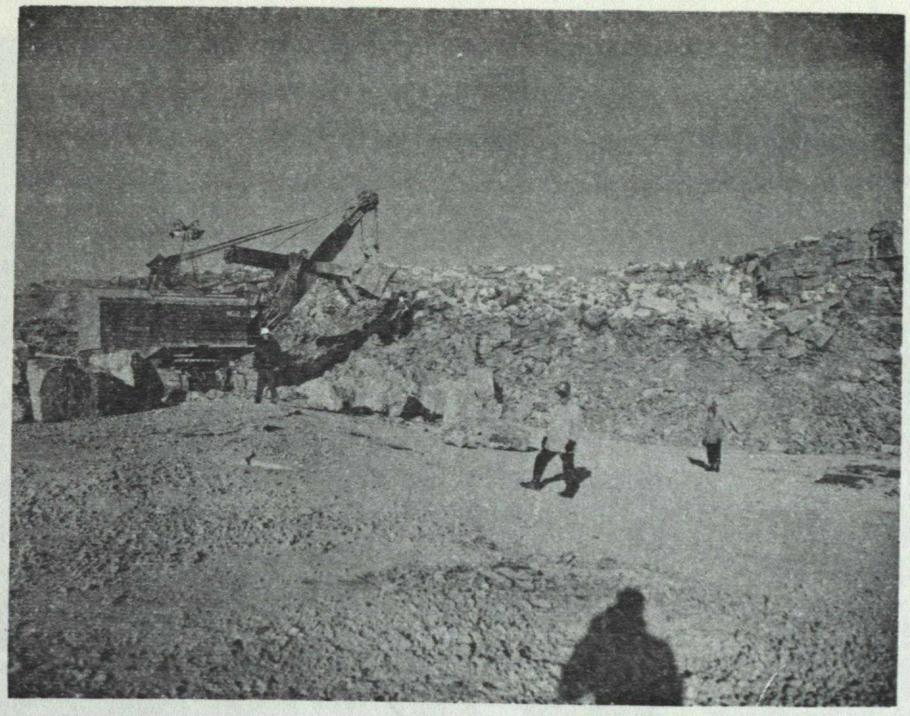

Fig. 3—Upheaval of floor of open pit.

considered either fixed or pinned. An intermediate case that is commonly used for square-ended compression members is assumed, giving an effective length of 800 ft. Using these assumptions and ignoring side restraint, it would require a horizontal stress of approximately 4000 psi to cause buckling and upheaval.

By taking into account the concentration of horizontal stress below the excavation, the horizontal field stress is deduced to be of the order of 2000 psi in the Paleozoic limestone. By changing any of the above assumptions, of course, the resultant answer can be changed considerably.

Horizontal Deformation of a Power Tunnel

The second case is concerned with two 51-ft-diameter tunnels which were excavated in horizontally stratified rock with approximately 175 ft of rock over the roof, plus 130 ft of soil. The tunnels are 5½ mi long. Excavation of the tunnels was by the heading and bench method.(1)

The bedrock formations are shown in Fig. 4. The rock types are mainly dolomite, limestone, sandstone, and shale. The formations contain bedding and slip planes and tend to be heterogeneous in composition. At the elevation of the tunnels the rock formations, in spite of the test results on core samples, were considered to be of low strength, with the exception of the massive Irondequoit limestone that forms the roof.

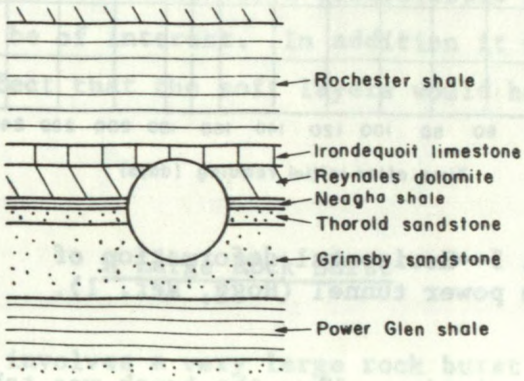

Fig. 4—Geological section at elevation of power tunnels (Hogg, Ref. 1).

The uniaxial compression strengths were as follows: Irondequoit limestone 15,000 psi, Reynales dolomite 12,000 psi, Thorold sandstone 15,600 psi, Grimsby sandstone 14,000 psi. The static moduli of deformation were as follows: Irondequoit limestone 7.28×10^6 psi, Reynales dolomite 5.95×10^6 psi, Thorold sandstone 2.62×10^6 psi, Grimsby sandstone 2.19×10^6 psi. It was not possible to obtain test values for the Rochester or Neagha shales as they disintegrated quickly on exposure to the atmosphere.

Measurements of the movement of the walls were begun during the excavation of the heading. Reference pins were anchored in the rock about 3 ft behind the working face. Horizontal diametral measurements were taken using an Invar micrometer tape.

The movement of the walls at one station in tunnel no. 1 are shown in Fig. 5. It can be seen that after an initial rapid movement the deformation rate decreased continuously until the excavation of the

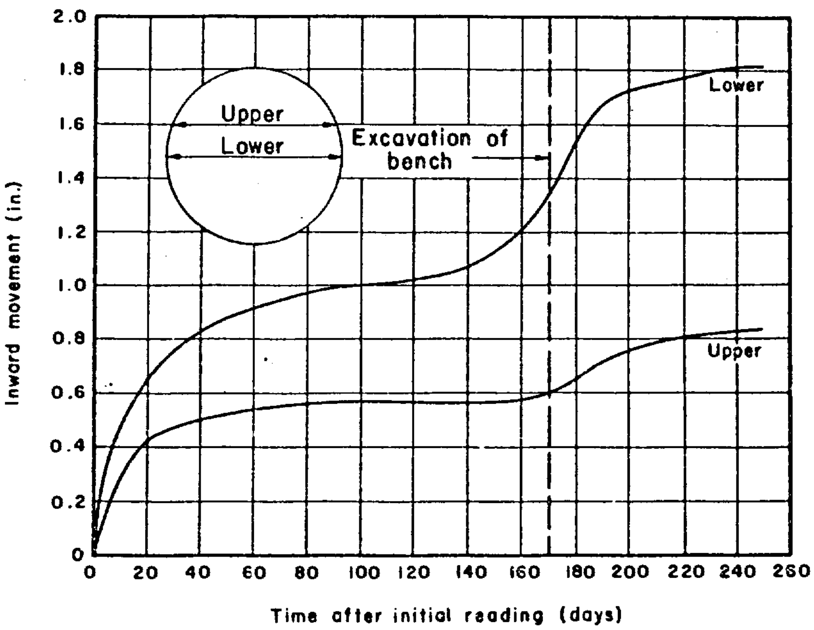

Fig. 5—Horizontal deformation of one power tunnel (Hogg, Ref. 1).

bench approached this station. When the bench was taken out, another large increment of deformation occurred followed by creep at a decreasing rate.

If the horizontal deformation resulting from the excavation of the heading or following the excavation of the bench is plotted against the logarithm of time, a straight line occurs.

Vertical movements in the tunnels were small. No measurements were made on the floor in tunnel no. 1; heaving, buckling, and cracking were observed, however. In tunnel no. 2, one measuring station indicated an upward movement of the floor of about 0.5 in. after the removal of the bench. Movements at the crown were of the order of 0.05 in.

The horizontal deformation of the walls of tunnel no. 2 varied from one-fourth to one-third of that measured in tunnel no. 1. The blasting and excavation of tunnel no. 2 caused the creep movements in tunnel no. 1 to increase for a period of time, after which they returned to the previous rate.

If it is assumed that the rock that has been deformed to produce the horizontal deformation of the walls has an average modulus of deformation of 5×10^6 psi, it is possible to calculate the order of magnitude of the residual stress in this area. By using an average vertical gravity stress of 340 psi and assuming that the horizontal deformation of 1.8 in. (see Fig. 5) is due to elastic strain, a horizontal stress normal to the axis of the tunnel of 4800 psi can be calculated. This, of course, is a very simple analysis.

Several questions arise from this trial analysis. Why, for example, did the second tunnel only have one-fourth to one-third of the deformation of the first tunnel? The determination of creep constants from the deformation-time measurements or a viscoelastic analysis, although complex, would also be of interest. In addition it would be interesting to determine the effect that the soft layers would have on this type of analysis.

A Large Rock Burst

The third case involves a very large rock burst in Northern Ontario which produced good seismic records at stations as far away as 920 km. It is possible that this rock burst may have released strain energy in the rock resulting from a high horizontal residual stress field.

The rock burst occurred in the mine shown in Fig. 6. Figure 6(a) is an elevation of the stoping area in the plane of the vein.[2] It can be seen that mining had proceeded to a depth greater than 4000 ft. Figure 6(b) is a cross section of the shaft showing the ore vein and two post-ore faults. Figure 6(c) is a plan of the vein and its relationship to the shaft at the 2450-ft level. Figure 6(d) is an idealized plan of the mined-out vein between the 1400- and 2800-ft levels.

The mineralization in this area is in pre-ore fault zones where the hangingwall has moved up 1300 to 1600 ft with respect to the footwall.[3,4] This faulting produced a considerable amount of gouge, brecciation, and alteration in the wall rocks. In addition, considerable post-ore cross and strike faulting has occurred.

Syenite porphyry is the most common host rock of the veins. Some augite syenite also occurs in the vein zone. Porphyry core samples were tested and gave the following mechanical properties: uniaxial

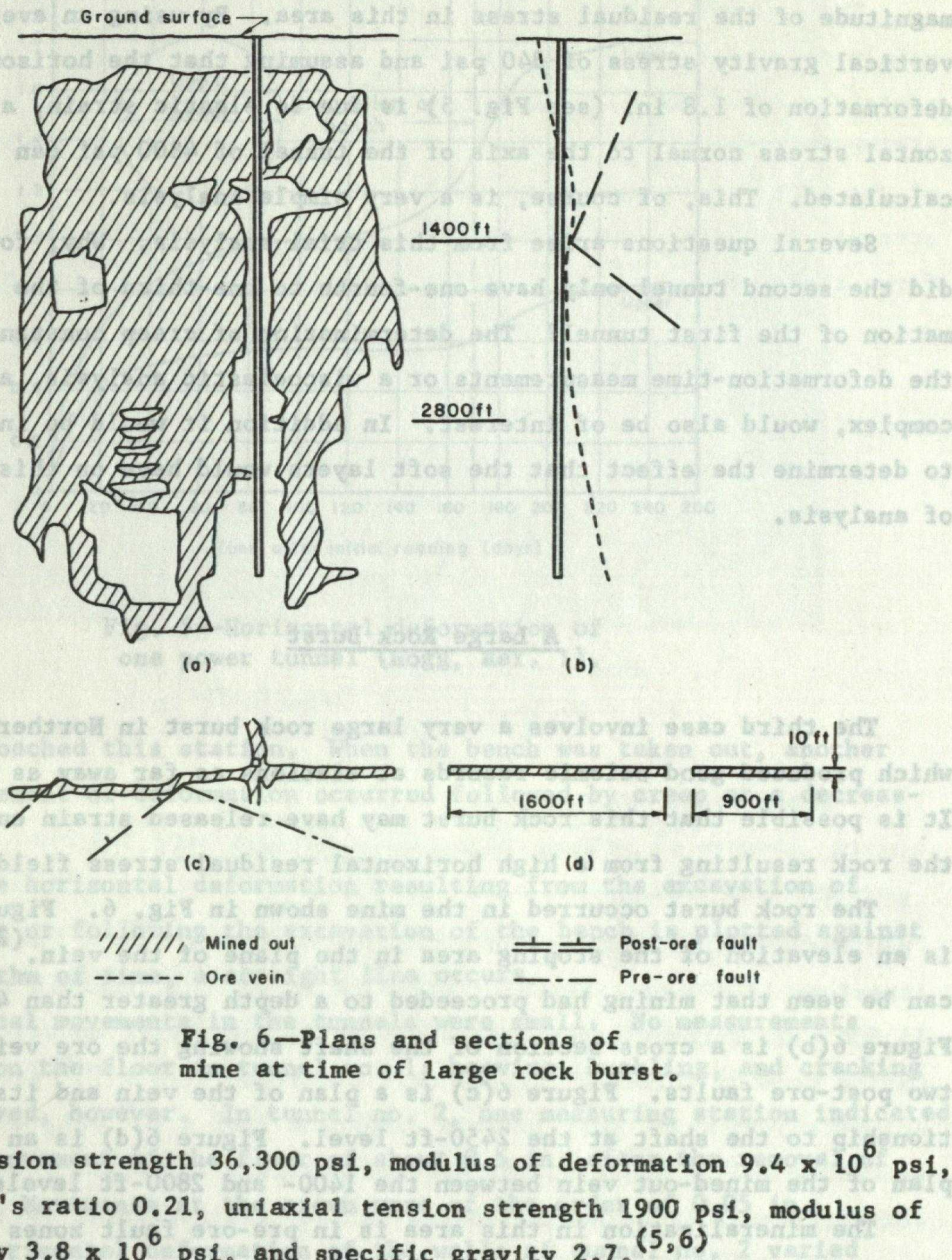

Fig. 6—Plans and sections of mine at time of large rock burst.

compression strength 36,300 psi, modulus of deformation 9.4×10^6 psi, Poisson's ratio 0.21, uniaxial tension strength 1900 psi, modulus of rigidity 3.8×10^6 psi, and specific gravity 2.7.[5,6]

The vein generally dips between 75 ft and 85 degrees. Its width varies from small stringers to a maximum of 70 ft. The vein rock consists of quartz and pyrite as well as the host rock.

During the driving of development drifts and crosscuts, many small bursts had occurred, producing falls of rock off the backs and walls. Neither the frequency nor the severity of these bursts increased with depth. They were particularly numerous, among other zones, on the 2825-ft horizon.[6]

When the large rock burst occurred, the shaft was closed completely from the 1400- to the 2800-ft level. By examining the seismic records recorded at Ottawa, Shawinigan Falls, Williams College, and Harvard University, the amount of energy transmitted to the ground surrounding the mine from the burst was estimated (using Jeffreys' method) at 5×10^{17} ergs.[7,8]

If the disturbance represents the strain energy released from the crushed pillar, the volume of ground included in the pillar would be of interest. With a height of 1400 ft, a width of 200 ft and an effective thickness of 50 ft, about 14×10^{6} cubic feet of rock might have been involved. This would be equivalent to a release of 2600 ft-lb of strain energy per cubic foot of rock.

On the other hand, if the disturbance represents the increased stress that is suddenly applied to the surrounding rock that must support the forces hitherto resisted by the rock in the pillar, the above calculation would not be valid.

The amount of strain energy that would be in the shaft pillar if only gravitational stresses were acting can be calculated. As the vein between the 1400- and the 2800-ft level is almost vertical, the horizontal field stress in this rock before mining, assuming no residual stress, would be about one-fourth of the vertical stress. The average horizontal stress in the pillar after mining would be increased by a factor of about 7.25, based on the width of the stopes.

The average strain energy in the pillar rock owing to this horizontal stress, as well as the vertical gravitational stress, would vary from 80 ft-lb/cu ft at the 1400-ft level to 300 ft-lb/cu ft at the 2800-ft level, with an average of about 160 ft-lb/cu ft. Consequently, at the depth of the burst, using the above assumptions regarding the disturbance, an addition to the gravity stresses, or a residual stress, of 2500 psi would be required to provide the seismic waves that were recorded. There are, of course, many questionable assumptions in these simple calculations.

Acknowledgments

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