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CREEP IN MODEL PILLARS

By Leonard Obert

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UNITED STATES DEPARTMENT OF THE INTERIOR

BUREAU OF MINES

1965

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By Leonard Obert

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PREFACE

This report is an elaboration and expansion of a paper titled "Deformation Behavior of Model Pillars Made From Salt, Trona, and Potash Ore" presented at the Sixth Symposium on Rock Mechanics, Rolla, Missouri, October 1964. A part of this research was supported by the Oak Ridge National Laboratory, and data from the Kansas salt tests (see figure 11) was supplied to the ORNL for independent analysis. This analysis was presented by R. L. Bradshaw, W. J. Boegly, and F. M. Empson at the same symposium and also published in the Proceedings.

This report incorporates the interpretations made by Bradshaw, Boegly, and Empson, and in addition includes data not previously published relating to the creep characteristics of Saskatchewan potash, which were supplied to the Bureau by Mr. Fred Wright, Lucius G. Pitkin, Inc., New York, N. Y. Reference is also made to the in situ behavior of salt pillars in a German potash mine. These results were supplied to the Bureau by Mr. F. W. Uhlenbecker, Hattorf Mine, Philippsthal, Germany.

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CREEP IN MODEL PILLARS

by

Leonard Obert¹

ABSTRACT

This Bureau of Mines investigation considers the deformational behavior (creep) of pillars made from three quasi-plastic rocks--salt, trona, and potash ore. The first phase of this study considers the design of a model pillar suitable for creep tests; in the second phase, six model pillars were prepared from salt from two sources, from trona, and from potash ore. The pillars in each group were subjected to a different but constant axial stress, and the axial strain was measured for 1,000 hours.

An analysis of the data shows that in general the creep rate for these model pillars can be expressed by the relationship

$$\dot{\epsilon} = K_1 \sigma_0^n$$

where $\dot{\epsilon}$ is the strain rate, σ_0 is the applied stress, and K_1 and n are constants. For the rocks included in this test, n ranged from 2.4 to 3.3.

INTRODUCTION

Unexcavated areas of rock left within the limits of a mine or system of underground openings are referred to as pillars and are the most important element in stabilizing the underground structure. In underground excavations in competent rock the pillars may remain intact and support the structure indefinitely; in operations involving induced caving, the pillars are generally reduced to a size such that they will crush and fail at some predetermined rate. In some incompetent rock, especially in salt and potash mines, the pillars undergo continuous plastic and/or viscoelastic deformation (creep). The creep rate depends on the average pillar stress, which in turn depends on the extraction ratio, and is usually limited to a value such that the pillar deformation and attendant effects will not interfere with mining operations.

A number of researchers have investigated the strength of model pillars where the limiting strength was determined from the load at which the model

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pillars ruptured or fractured with a sudden release of applied load; that is, as a brittle material. The typical uniaxial compressive test specimen is in effect a model of a prototype pillar having the same scaled dimensions. A number of empirical relationships have been developed that relate specimen strength to specimen size and shape (height-to-width or diameter ratio). In these investigations the specimen strength also depends on other factors, especially the rate of loading and the end constraint; that is, the resistance offered by the bearing plate to the lateral deformation of the specimen.

In considering the mechanical properties of materials, it is generally assumed that failure depends not only upon brittle fracture but, for more ductile materials, on some limiting strain or strain rate. Although the magnitude of the strain or strain rate that limits the utility of the mine pillar is not a unique value but rather depends on the system of mining and other factors, it is evident that any rational basis for designing a mining operation in a rock that exhibits plastic or visco-elastic effects would require a knowledge of the deformational characteristics of supporting pillars.

The objective of this investigation is directed toward this end. In particular, the investigation considers the deformational behavior of model pillars made from salt, trona, and potash ore (sylvite plus halite) tested under a constant applied load. Although the load on the mine pillars is not constant at the time it is being formed, it is virtually constant over the larger part of its effective lifetime. As the load in the pillar prior to reaching this constant value is less than that after the pillar has assumed its maximum load, any design based on constant load conditions would be on the conservative side.

FACTORS AFFECTING THE DESIGN OF A MODEL PILLAR FOR CREEP TESTS

Preliminary to the development of a model pillar for creep tests, a limited investigation was made to determine the effects of specimen shape and end constraint on the strength and deformational behavior of salt and trona specimens subjected to moderate rates of loading (approximately 100 psi/minute). In figure 1 the average axial strain versus axial stress results are presented for a set of three dimensionally identical salt specimens having a diameter-to-height ratio of 1 ($D/H = 1$), tested without any friction reducer (lubricant) between the ends of the specimen and bearing plates, and loaded at a rate of 100 psi/minute (approximate test time, 1 hour). These results showed a relatively elastic behavior up to an axial stress of 2,500 psi and a very rapidly increasing yield above this value. At an axial stress of 5,500 psi the specimens failed suddenly like a brittle material. An equivalent set of three specimens were tested under the same conditions except that a friction reducer, consisting of an 0.006-inch-thick Teflon² film greased on both sides, was placed between the specimen ends and bearing plates. The axial stress-strain results were approximately the same as those obtained without friction reducer, but the specimens failed at a lower value, 4,700 psi.

²Reference to specific brands is made for identification only and does not imply endorsement by the Bureau of Mines.

FIGURE 1. - Salt-Stress Versus Strain.

FIGURE 2. - Salt-Effect of Specimen Shape and End Constraint.

Correspondingly, groups of three dimensionally identical specimens for which $D/H = \frac{1}{2}$, $D/H = 2$, and $D/H = 4$ were tested with end lubricant. These specimens failed by fracture with a sudden release of applied load. The $D/H = 2$ and $D/H = 4$ specimens failed at a value only slightly greater than that for the $D/H = 1$ specimens, as indicated by curve a, figure 2, in which C_1 is the compressive strength of a $D/H = 1$ specimen, and C_x is the compressive strength of $D/H = \frac{1}{2}$, 1, 2, and 4 specimens. Lateral flow tended to split the $D/H = 2$ and $D/H = 4$ specimens into several pieces, each of which probably failed more or less like $D/H = 1$ specimens. Groups of three dimensionally identical $D/H = \frac{1}{2}$, $D/H = 2$, and $D/H = 4$ specimens were also tested without end lubricant. The $D/H = 2$ specimens did not fail as a brittle material. Rather, at the maximum recorded load they failed slowly but by fracture with a complete release of applied load. The $D/H = 4$ specimens did not fail by fracture, but the creep rate became so large that the test was discontinued; the axial strain at the conclusion of this test was almost 50 percent. The results of these tests are shown in curve b, figure 2, where the values of the applied stress are the maximum reached during the course of the test. For purposes of comparison, the D/H versus compressive strength relationship for brittle rock specimens is shown in curve c, figure 2 (6),³ the empirical equation for which is

$$C_x = C_1 (0.778 + 0.222 D/H).$$

Several 8-inch by 4-inch-diameter specimens from the same salt were loaded triaxially with radial loads between 10 and 20 percent of the axial load. These specimens did not fail by fracture but deformed to the point that the rubber confining jacket was ruptured, and the test was terminated.

These preliminary tests indicate two significant facts; (1) for a quasi-plastic material the end constraint strongly affects the specimen strength, and (2) as D/H increases, the specimens lose their brittle characteristics and tend to flow rather than to fracture. Because the deformational behavior of quasi-plastic materials is so strongly dependent on the end constraint, model pillars made from these materials should provide some means of controlling and determining the magnitude of this factor. The model shown in figure 3 was found to satisfy this requirement. $D/H = \frac{1}{2}$, 1, 2, and 4 models were prepared from 5-5/8-inch-diameter cores in which the central peripheral part was removed with a cylindrical grinder to the dimension shown in figure 3. Steel rings were cemented to the ends of each model with an epoxy cement that completely filled the small gap between the rock and the ring. The ends of the model were allowed to extend one-eighth inch past the steel ring so that when it was loaded no axial force was applied to the steel rings. Three resistance strain gages oriented to respond to tangential strain were cemented to the periphery of each steel ring at 120° intervals. The internal pressure applied to the ring, P_1 , was determined from the measured (average) strain, ϵ , by

$$P_1 = \frac{E \epsilon (b^2 - a^2)}{2 a^2},$$

³Underlined numbers in parentheses refer to items in the list of references at the end of this report.

FIGURE 3. - Model Pillar.

FIGURE 4. - Axial Versus Radial Stress.

where a and b are the inside and outside radii of the rings, and E is Young's modulus for steel ($E = 30 \times 10^6$ psi). If it is assumed that the lubricant reduces the end constraint to a negligible value, then the radial stress, σ_r , in the end of the model is:⁴

$$\sigma_r = -P_1.$$

The constraint produced by the rings was found by experiment to depend on both the axial load and the D/H ratio. An average axial versus radial-stress result for three $D/H = 2$ model pillars made from salt, loaded at a rate of 100 psi/minute is shown in curve a, figure 4. In the elastic range (up to 2,500 psi) the constraint offered by the rings was small, but above this value it was almost proportional to the applied axial stress, and at an applied stress of 9,000 psi the radial stress was approximately 66 percent of this value. A similar result was obtained for a group of three $D/H = 4$ model pillars loaded at a rate of 250 psi/minute as shown in curve b, figure 4.

⁴Where compression is negative.

FIGURE 5. - Trona-Mohr's Circles and Envelope.

FIGURE 6. - Trona-Stress Versus Strain.

The average strengths of groups of three dimensionally identical model pillars, for which $D/H = \frac{1}{2}, 1, 2,$ and 4 , are presented in curve d, figure 2. These models were tested at a rate of 250 psi/minute with end lubricant. The mode of failure in these model pillars was virtually the same as that for the specimens having the same D/H ratio tested without end lubricant. Thus, the pillar models give a strength result comparable to that obtained with a conventional compression specimen. The steel rings provide a means of determining and controlling the magnitude of the end constraint.

Similar results were obtained for model pillars made of potash except that both the conventional specimens and the model pillars gave somewhat lower strength values. However, trona specimens, prepared in the same manner and tested under the same conditions, behaved more nearly like an elastic brittle material. The Mohr's circles and envelope for a group of 8-inch by 4-inch-diameter trona specimens (fig. 5) are indicative of a brittle material. The axial stress-strain curve for a $D/H = 1$ specimen tested without lubricant is shown in figure 6. This result can be compared with the corresponding result for salt, figure 1. The average strength results for groups of three $D/H = \frac{1}{2}, 1, 2,$ and 4 specimens tested without lubricant are given in figure 7. Except for the $D/H = 4$ specimens and model pillars, each test terminated in a brittle failure; the $D/H = 4$ specimens failed slowly by fracturing, allowing the applied load to decrease almost to zero. The radial stress in the end of $D/H = 4$ model pillar is presented in curve c, figure 4. This curve also reflects the more nearly elastic behavior of trona.

FIGURE 7. - Trona—Effect of Specimen Shape.

As the D/H ratio is increased in the more plastic materials, the tendency for the models to flow rather than to fracture increases; salt and potash model pillars loaded at the rate of 250 psi/minute did not fail with a loss of applied load, but rather deformed almost like an elasto-plastic material.⁵ Two $D/H = 4$ pillars of salt were loaded to 16,000 psi in less than 10 seconds. These pillars did not fail by sudden fracture, although there was some slough from the pillar

⁵An elasto-plastic material is one in which the strain is proportional to the stress up to the yield point, but at the yield point the material will strain without limit.

surfaces. This result suggests that if the pillars in a salt or potash mine have a diameter-to-height ratio of 4 or greater⁶ they will not fail by sudden fracture, regardless of the rate of loading. Thus, in this type of mine the possibility of a chain reaction type of collapse is excluded; that is, the collapse of an extensive area initiated by the sudden failure of one pillar which transfers its load to an adjoining pillar causing it to fail.

EQUIPMENT AND TEST PROCEDURE

On the basis of the preliminary tests described in the foregoing section, a model pillar was designed for creep tests. The model was identical to that shown in figure 3, with a D/H ratio of 4 and a constraining ring thickness (b - a) of 3/4 inch, (fig. 8). In addition to the three resistance strain gages on each constraining ring, two dial gages with 0.0001-inch scaled divisions were attached at diametrically opposite points to the inner surface of the rings, to indicate the axial deformation of the pillar. The pillar model was loaded in a 250,000-pound hydraulic compression testing machine. The hydraulic ram on the testing machine was O-ring sealed, and when the hydraulic system was closed with a precision valve it maintained a constant load on a steel (noncreeping) specimen indefinitely, providing that the temperature was kept constant. Six testing machines were employed in these tests.

Six model pillars were prepared from each of the following: Salt from Detroit, Mich., and from Lyons, Kans.; trona from Green River, Wyo.; and potash ore (sylvite plus halite) from Carlsbad, N. Mex. A model was placed in a hydraulic testing machine, and greased Teflon sheets (friction reducers) were inserted between the bearing plates and the ends of the model. To seat the bearing plates the model was rapidly loaded to approximately 75 percent of its estimated compressive stress and then unloaded. Initial no-load dial gage and strain gage readings were recorded. The specimen was loaded to some predetermined value in as short an interval as possible, usually less than 10 seconds. The applied load was maintained at this value throughout the test, which was continued for 1,000 hours, or until the axial pillar strain exceeded 25 percent, or until the specimen failed. During the first few hours of this period it was necessary to adjust the applied load almost continuously to keep it constant (due to creep in the specimen). After this initial period the load was adjusted less frequently, and after 48 hours it was necessary to adjust the load only twice daily to maintain a constant value within ± 2 percent. The equipment was kept in an air-conditioned room in which the temperature was constant so that the load change did not exceed the ± 2 percent limit.

The dial gage and the resistance strain gages on the constraining rings were read each time the load was adjusted, so that at least two readings were taken each 24 hours throughout the test period.

⁶The cross sections of mine pillars are seldom circular. However, it has been found from model studies that the relationships between compressive strength and diameter-to-height or width-to-height ratio are virtually identical, where the width of a pillar is its smaller lateral dimension.

FIGURE 8. - Model Pillar Mounted in Testing Machine.

TEST RESULTS

The radial stress developed in the ends of the models by the constraining rings increased with the axial stress. For example, in figure 9 for Michigan salt, for an axial stress of 2,000 psi, the radial stress was 18.5 percent of the axial stress and it remained virtually constant throughout the test period. At an axial stress of 4,000 psi, the radial stress increased to approximately 40 percent of the axial stress at 500 hours, and then remained essentially constant for the remainder of the test. For axial loads of 6,000, 8,000, and 10,000 psi, the radial stress was approximately 50 percent of the axial stress throughout the test period, except that in the specimen loaded at an axial stress of 8,000 psi the radial stress became erratic after 500 hours. This erratic behavior is believed to be due to stress readjustments caused by internal fracturing in the model. The results in figure 9 also indicate the transition in this salt from an elastic to a quasi-plastic condition at a

FIGURE 9. - Michigan Salt-Ratio, Radial to Axial Stress Versus Time.

FIGURE 10. - Trona-Ratio, Radial to Axial Stress Versus Time.

FIGURE 11. - Kansas Salt-Strain Versus Time.

FIGURE 12. - Michigan Salt-Strain Versus Time.

stress level above 3,000 psi. The radial stress in the ends of the Kansas salt and New Mexico potash models was essentially the same as that for Michigan salt. However, in the Wyoming trona models the radial stress was not as large (fig. 10) ranging from about 20 to 25 percent for an axial stress of 8,000 psi to 30 to 35 percent for an axial stress greater than 10,000 psi. The trona results are somewhat more variable, due to a greater specimen-to-specimen variation in this rock type.

The axial strain⁷ versus time results are given in figures 11, 12, 13, and 14. Except that the strength, and hence the largest axial stress, was not as great for potash as for salt, the Michigan and Kansas salt and the New Mexico potash results show a general similarity. When a model was loaded, there was an initial elastic deformation combined inseparably with a high strain (creep) rate; the strain rate decreased with time, becoming in most instances virtually constant after 500 to 600 hours. Except for the potash models loaded at 2,000 and 3,000 psi, the strain at any

⁷Axial strain calculated from $\Delta H/H$, where H is the initial axial length (height) of the pillar.

FIGURE 13. - New Mexico Potash—Strain Versus Time.

FIGURE 14. - Wyoming Trona—Strain Versus Time.

time increased with the load. The potash exception resulted from the initial strain in the model loaded at 2,000 psi exceeding that produced in model loaded at 3,000 psi. This anomalous result is attributed to specimen variation.

The test of the Michigan salt models loaded at 10,000 and 12,000 psi and the Kansas salt model loaded at 10,000 psi were terminated after the strain exceeded 25 percent. None of the salt and potash models failed like a brittle material by sudden fracture with a decrease in load-bearing capacity. Also, these models did not exhibit a tertiary phase; that is, a terminal phase in which the strain rate accelerated. The models loaded at low stress levels showed no discernible indication of failure (spall) on the pillar surface. At intermediate stress levels the surface bulged and there was some spall; at high stress levels there was heavy spall in all instances. However, when the latter group of specimens were wedged apart at the mid-plane (fig. 15), it was found that the diameter of the solid core was at least 94 percent of the original diameter.

The trona specimens were tested at intermediate and high axial loads.

FIGURE 15. - Core of Model Pillar After 1,000-Hour Creep Test.

At intermediate loads (8,000 to 10,000 psi) the creep curves were similar to those for potash except that the magnitude of the strain at any time was much smaller. At high loads (12,000 to 14,000 psi) the models failed by slowly crushing with a substantial reduction in the applied Load. No attempt was made to reload these models. The models loaded at 12,000 and 13,000 psi showed a well-defined tertiary phase. The creep curves for the models loaded at 10,000, 11,000, 12,000, and 13,000 psi are not in an ascending order, a result that also was attributed to specimen variation.

ANALYSIS OF RESULTS

The creep curves for the model pillars, presented in figures 11, 12, 13, and 14, are similar to creep curves obtained by other investigators for different-shaped models tested under a variety of conditions and suggest that a general expression of the form

$$\epsilon = A + Bt + Cf(t) \quad (1)$$

can be fit to the data. It was found that if logarithmic or power-law functions were used for $f(t)$, equation (1) would not fit the data. However, if $(1 - e^{-\alpha t})$ was used for $f(t)$, values of A , B , C , and α could be chosen so that equation (1) could be fit to any one of the creep curves (except for the curves exhibiting a tertiary phase, and for $t < 24$ hours), for example, as shown in figure 16. This fact implies that the pillar models may behave like a Burger's body (fig. 17), the general expression for which is

FIGURE 16. - Michigan Salt—Strain Versus Time.

FIGURE 17. - Model for Burger's Body and Corresponding Creep Curves.

$$\epsilon = \frac{\sigma_o}{E_m} + \frac{\sigma_o}{3\eta_m} t + \frac{\sigma_o}{E_k} \left[1 - e^{-E_k t / 3\eta_k} \right]. \quad (2)$$

Equation (2) is identical to equation (1) if

$$A = \frac{\sigma_o}{E_m},$$

$$B = \frac{\sigma_o}{3\eta_m},$$

$$C = \frac{\sigma_o}{E_k}$$

and $f(t) = (1 - e^{-E_k t / 3\eta_k})$.

In equation (2) σ_o/E_m is the elastic strain, $\frac{\sigma_o}{E_k} (1 - e^{-E_k t / 3\eta_k})$ is the initial or transient creep, and $\frac{\sigma_o}{3\eta_m}$ is the steady-state creep. The slope of the steady-state phase of the creep curve (that is, constant strain rate) is $\frac{\sigma_o}{3\eta_m}$.

Although, as previously stated, equation (1) can be fit to any given curve by the proper selection of A, B, C, and α , it was found that no set of constants would provide a fit for the family of curves for any rock type, primarily because $3\eta_m$ was not constant for different values of σ_o . In fact, for a family of curves it was found that the steady-state strain rate can be expressed by

$$\dot{\epsilon}_m = D \sigma_o^n. \quad (3)$$

In figures 18, 19, 20, and 21 the relationship between the strain rate and the applied load is given for the four rock types, and the least-square slopes of the curves are given in table 1.

TABLE 1. - Exponent n for salt, potash, and trona

Rock type	Load range, psi	n
Salt (Kansas).....	4,000-10,000	3.0
Salt (Michigan).....	2,000- 8,000	3.1
Potash (New Mexico).....	2,000- 7,000	3.3
Trona (Wyoming).....	8,000-12,000	2.4

A comparison of the results obtained in this investigation with those obtained by other researchers is possible in a few instances. A power-law relationship between strain rate and applied stress has been reported by other investigators (7). Kendall (4) found that salt specimens tested under

FIGURE 18. - Kansas Salt-Strain Rate Versus Axial Stress.

FIGURE 19. - Michigan Salt-Strain Rate Versus Axial Stress.

conditions similar to those in this investigation gave a value of $n=1.9$. In table 2 the creep rate obtained by Griggs (2) for a single salt crystal loaded in uniaxial compression to about 900 psi is compared with creep rates for model pillars of Michigan salt, Kansas salt, and New Mexico potash loaded at 1,000 psi as determined by extrapolation. The creep rates obtained in the model pillars are more than one order of magnitude less than the one obtained by Griggs. This lower creep rate may be a result of the higher lateral constraint in the model pillars. The results from the model pillar test show that the coefficient of viscosity, η_m depends on the applied axial stress, and probably on the state of stress. The magnitude of η_m can be determined from the slope of any one of the steady-state creep curves from the relationship

$$\eta_m = \frac{\sigma_o}{3 \cdot \dot{\epsilon}_m} . \text{ For example, for the 6,000-psi creep curve for Kansas salt, } \dot{\epsilon}_m = 2.75 \times 10^{-5} \text{ hr}^{-1}, \text{ or } 7.6 \times 10^{-9} \text{ sec}^{-1}. \text{ Thus, } \eta_m = \frac{6,000}{3 \times 7.6 \times 10^{-9}} =$$

$2.6 \times 10^{11} \text{ lb sec/in}^2$. The value given by Griggs for a single salt crystal loaded at ~ 900 psi is $\eta = 10^{11} \text{ lb sec/in}^2$.

FIGURE 20. - New Mexico Potash—Strain Rate Versus Axial Stress.

FIGURE 21. - Wyoming Trona—Strain Rate Versus Axial Stress.

TABLE 2. - Strain rates for salt, potash, and trona

	Axial pillar stress	Strain rate, $\dot{\epsilon}$ - sec ⁻¹
Single salt crystal (Griggs).....	900	10 ⁻⁹
Michigan salt, model pillar.....	¹ 1,000	1.0 x 10 ⁻¹⁰
Kansas salt, model pillar.....	¹ 1,000	0.3 x 10 ⁻¹⁰
New Mexico potash, model pillar.....	¹ 1,000	0.4 x 10 ⁻¹⁰

¹ Extrapolated.

Höfer (3) measured the in situ lateral deformation in a number of potash pillars over periods up to 500 days. A typical creep curve is shown in figure

FIGURE 22. - Lateral Pillar Deformation Versus Time From Höfer (3).

22. In general, his results show that the in situ strain rate is constant for any specific average pillar stress. Höfer calculated the average pillar stress from the deadweight pillar load⁸ and presented the creep rate versus average pillar stress results graphically (3, fig. 46). These data are reproduced in figure 20. This representation shows that Höfer's in situ data also follow a power-law curve of the form $\dot{\epsilon}_m = D \sigma_o^n$, but that $n=6.3$, as compared with $n=3.3$ for New Mexico potash. The in situ strain rates were more than an order of magnitude less than those obtained from model pillars made from New Mexico potash. These differences may be due to any or all of several factors. Höfer measured the lateral deformation (not axial deformation) in rib-type pillars having a height-to-width ratio of 1-to-1 to 1-to-2. Also, the mineralogy of the German potash is not equivalent to that of the New Mexico potash. Axial pillar deformation measurements taken in the Hattorf mine, Philippsthal, Germany, over an 8-year period also show that in areas undisturbed by mining the creep rate is virtually constant.

The data in figure 11 were independently analyzed by Bradshaw, Boegly, and Empson (1). The slope of the strain-time curves for each axial stress value in figure 11 was plotted on log-log coordinates (fig. 23). These data plot as approximately straight lines with a common slope of -0.6. Hence, the equation for the strain rate as a function of time can be expressed by

⁸Average pillar stress is given by $\sigma_{p(av)} = \sigma_v \left(\frac{1}{1-R} \right)$ where $\sigma_{p(av)}$ is the average pillar stress, σ_v is the vertical stress ($\sigma_v = \gamma h$, where γ is the unit weight of the rock and h is the vertical depth), and R is the extraction ratio.

FIGURE 23. - Creep Rate Data on Model Pillars Made From Kansas Salt.

$$\dot{\epsilon} = K_1 t^{-0.6}. \quad (4)$$

Also, from equations (3) and (4) the strain rate can be expressed as a power-law function of both the applied stress and time, that is,

$$\dot{\epsilon} = K_2 \sigma_0^n t^\beta, \quad (5)$$

where $n = 3.1$, $\beta = -0.6$, and K_2 is the ordinate intercept. The creep rates for Michigan salt, New Mexico potash, and trona do not plot as straight lines on log-log coordinates; therefore, the creep rates for these rock types do not vary as a power-law function of time. However, using the same test procedure described herein, Wright measured the creep in model pillars made from various grades of Saskatchewan potash ore. These data plot as straight lines on log-log coordinates, figures 24 and 25. Hence, the strain-time relationship is of the form

$$\epsilon = K_3 t^\alpha, \quad (6)$$

where α has the values given in table 3. Also, by differentiation of equation (6),

$$\dot{\epsilon} = K_4 t^{\alpha-1} = K_4 t^\beta, \quad (7)$$

where the values of β are given in table 3.

Thus, the creep behavior of the Saskatchewan potash ores is similar to that of the Kansas salt except that the values of the negative exponents are larger.

FIGURE 24. - Creep Rate in Model Pillars Made From Saskatchewan Potash Ore, Axial Stress 4,000 psi.

TABLE 3. - Creep rate exponents for Saskatchewan potash ore

Axial stress, psi	Halite or low-grade ore		Average-grade ore		High-grade ore	
	α	β	α	β	α	β
4,000.....	0.11	-0.89	0.16	-0.84	0.26	-0.74
5,000.....	.15	-.85	.22	-.78	.27	-.73

CONCLUSIONS

The objectives of this investigation were twofold, to develop a model pillar realistically related to its prototype in the mine, and to study the strength and deformational behavior of model pillars made from salt, potash, and trona.

FIGURE 25. - Creep Rate in Model Pillars Made From Saskatchewan Potash Ore, Axial Stress 5,000 psi.

The first of these objectives is reasonably satisfied by the pillar model shown in figure 3. This model has an advantage over conventional cylindrical or prismatic specimens in that there is no discontinuity in the model material at either the top (roof) or bottom (floor) of the pillar. The steel constraining rings make it possible to measure the constraining stress in the ends of the pillar. In creep tests (pillar subjected to constant load) the constraining stress developed almost immediately after application of the pillar load and remained relatively constant throughout the test period at a value ranging from 20 to 50 percent of the axial stress. In relation to the radial stress in the ends of mine pillars, the value measured in the model pillars may be low because the coefficient of the friction reducer may not be zero and because there may be a compressive stress in the roof and floor of mine openings which, by superposition, would add to the constraining stress developed at the pillar ends. In situ stress measurements (5) made in the core of limestone pillars of similar shape indicate that the radial stress ranges from 30 to 70 percent of the axial stress, probably it would be higher in more plastic rocks. The strength of model pillars made of brittle rock increases with the radial stress. How the creep rate is affected by radial stress was

not determined, but intuitively the creep rate would not be expected to decrease with increasing stress rate. Hence, from an engineering standpoint estimates of strength or creep rate based on model pillar tests should be on the conservative side.

Although field data are meager, there is a general agreement between model pillar and in situ pillar measurements. From all indications the creep rate in salt and potash model pillars and mine pillars becomes constant after a relatively short transient period. Also, the creep rate in both model and mine pillars varies as some power of the applied pillar load. However, there appears to be a significant difference in the exponent of the pillar stress determined from model and in situ tests. Additional in situ data will be required to determine if this difference is real. The fact that the creep rate can be expressed as some power of the applied load, that is,

$$\dot{\epsilon}_m = D \sigma_o^n,$$

has a special engineering significance because the in situ pillar stress can be reasonably approximated from the depth and extraction ratio. Hence, this relationship can be used for design purposes.

Attempts to fit the pillar model data for a given rock type to a simple (four-element) rheological model were not successful. However, because the creep rate becomes relatively constant after a short period (500 to 600 hours) compared with the lifetime of most mine pillars, the transient phase has little importance in relation to the evaluation of mining problems.

The salt and potash models did not exhibit any tertiary phase (accelerated creep rate), at strains up to 25 percent. Hence, using accelerated strain rate as a criterion of impending failure would not be justified. However, trona exhibits a well-defined tertiary phase, and for this rock type accelerated creep rate could probably be used to forewarn of pillar failures.

Because evaporite minerals tend to creep rather than to fracture when the lateral constraint is high enough, if the width-to-height pillar ratio in mine pillars is sufficiently large (4 or greater for salt and potash) the possibility of a chain-reaction failure is excluded.

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⁹Titles enclosed in parentheses are translations from the language in which the item was published.