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PLATE-LOAD TESTING ON ROCK FOR DEFORMATION AND STRENGTH PROPERTIES

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MINING RESEARCH CENTRE

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Plate-Load Testing on Rock for Deformation and Strength Properties

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ABSTRACT: Plate-load tests were conducted underground on rock to determine *in situ* strength and deformation properties. The plate-load test provides information on the rock mass properties as opposed to the strength of the rock substance. When a plate is placed on the surface of the material to be tested and the contact pressure increased, the plate deflects as the material deforms, and shear failure of the material ultimately occurs.

Tests were conducted on three different materials: iron ore, paint rock, and ash rock. The results were compared, in some cases, with the results of laboratory testing and, in other cases, with the results of analyzing failures of the rock mass. Moderately good agreement was obtained between these independent methods of determining the material properties. However, the principal aspect that emerges is the dispersion of strength values that must be expected in testing geological materials.

Suggested specifications are given for plate-load testing, conventional uniaxial compression testing, and classification uniaxial compression testing.

KEY WORDS: plate-load tests, rock mechanics, rock (material), strength, deformation, shear strength, bearing strength, brittle fracture, specifications

The mechanical properties of a rock mass depend on the nature of the rock substance, stratigraphy, and structural features. Laboratory tests are generally conducted on specimens of the rock substance, which therefore give no information on the effects of stratigraphy or structural features. Special laboratory studies have been made in the past on these other factors, but no substantiation has been obtained that the mass properties can be predicted from such testing. Consequently, the purpose of the testing described in this paper was to determine the deformation

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and strength properties of the rock mass in various formations in which instability was being experienced.

Theory

Deformability

Uniform Pressure—The basis of the plate-load test is that if a plate is placed on an extensive surface, equivalent to a semiinfinite half space, and the contact pressure is increased, the resulting settlement is a function of the effective modulus of deformation of the subgrade. The settlement of a bearing area on which a uniformly distributed pressure is acting has been solved in the form of the general Eq 1:

$$d_c = q\,B\,I/E \qquad (1)$$

where:

d_c = deflection of corner of loaded area,
q = uniformly distributed pressure,
B = width of loaded area,
I = influence value varying with ratio of length to breadth of bearing area as well as Poisson's ratio of the subgrade, and
E = modulus of deformation of ground.

For a square area the settlement of the center point, d_o, is twice that of the corner settlement.

If Poisson's ratio, μ, is 0.3, the settlement of the center point of a square bearing area is approximately

$$d_o = q\,B/E \qquad (2)$$

For values of Poisson's ratio other than 0.3, the adjustment can be made by knowing that the settlement varies directly as $(1 - \mu^2)$.

With the above equations, providing all the appropriate conditions are fulfilled, it is then possible to determine E of a rock mass by measuring the deflection at either the center or corner of a square area loaded with uniformly distributed pressure on the rock surface.

Rigid Bearing—For a rigid foundation resting on a semiinfinite elastic body, the equation for a rigid circular bearing can be used [*1*]:[3]

$$d = \frac{Q(1 - \mu^2)}{2RE} \qquad (3)$$

where:

Q = load on the foundation and
R = radius of the bearing area.

[3] The italic numbers in brackets refer to the list of references appended to this paper.

The contact pressure for this rigid, circular bearing area would be [*1*]:

$$\sigma_v = \frac{Q/(\pi R^2)}{2(1 - (r/R)^2)^{1/2}} \quad (4)$$

where r = distance from center of bearing area.

A similar equation for a long, rigid foundation of width B has been derived as follows [*1*]:

$$\sigma_v = \frac{Q/B}{\pi(0.25 - (x/B)^2)^{1/2}} \quad (5)$$

where x = distance from center of foundation. Figure 1 shows the pattern of pressure variation.

Using a square bearing area for the prediction of settlement or for

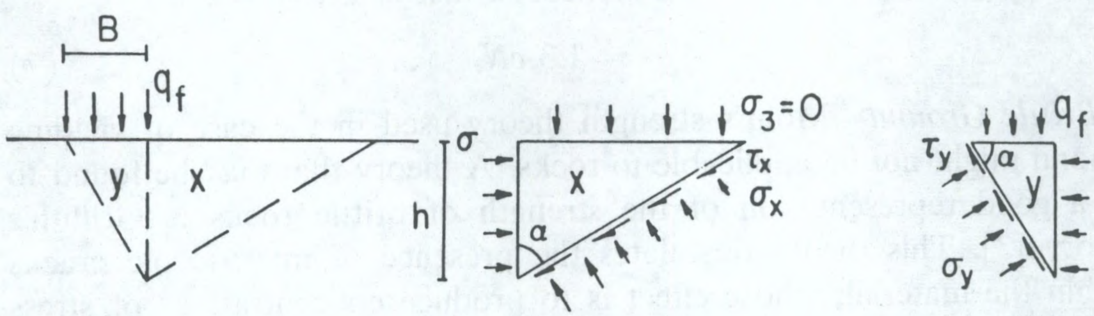

FIG. 1—*Wedge analysis of stresses.*

the determination of the modulus of deformation of the subgrade, it is probable that the practical device of replacing R with a B that represents a square of equal area to the circle would be satisfactory.

Strength

Yielding Ground—An alternate function of the plate-load test is based on the assumption that, if the contact pressure of a plate on an extensive surface of a material to be tested is increased, shear failure of the material will ultimately occur. The results of such tests must then be interpreted, using a bearing capacity theory to calculate fundamental strength parameters. With such fundamental strength parameters it is then assumed that the test data can, in effect, be extrapolated to other geometry and other loading conditions for the prediction of failure pressures.

For yielding ground the bearing capacity theory developed and substantiated in soil mechanics can be used [2]. The bearing pressure at failure, q_f, is related to the fundamental strength parameters through the following equation:

$$q_f = 0.5\gamma B N_\gamma + c N_c + p N_p \qquad (6)$$

where:

γ = density of ground,
B = width of a long footing,
c = cohesion, as in Mohr's strength theory,
p = surcharge pressure on ground surface adjacent to bearing pressure, and
N_γ, N_c, N_p = bearing capacity factors, functions of internal friction angle ϕ [2].

Equation 6 could be used for a plate-load test which was performed on a horizontal surface with a long plate. If the test is run on a wall or a vertical ground surface, then the first and third terms no longer contribute to the ultimate bearing capacity. In addition, if the plate is square the coefficient of c must be increased from 1 to 1.3 [2]. Consequently, the equation for a square plate on a vertical surface is as follows:

$$q_f = 1.3\, c N_c \qquad (7)$$

Brittle Ground—Mohr's strength theory used in the case of yielding ground might not be applicable to rocks. A theory that may be found to be a good representation of the strength of brittle rocks is Griffith's theory [3]. This theory postulates the presence of microscopic cracks within the material, whose effect is to produce concentrations of stress around their boundaries.

If the principal stress in the rock is tension and normal to the crack, then a tensile stress many times this average stress will be created at the ends of the cracks. If the principal stress is compression, then tension can be produced in a direction at right angles to the direction of the compressive strength.

When the stress concentrations are equal to the tensile strength of the material, cracks will be propagated. As the length of the crack transverse to the field stress increases, the stress concentrations become greater; consequently, it is visualized that once initiated, the propagation of the crack will lead to failure of the material.

By assuming that the cracks are elliptical in shape and that they are randomly oriented, the following criteria for failure have been established [4]:

$$\sigma_3 = -T_s \quad \text{when} \quad \sigma_1 + 3\sigma_3 < 0 \qquad (8)$$

$$\frac{(\sigma_1 - \sigma_3)^2}{\sigma_1 + \sigma_3} = 8T_s \quad \text{when} \quad \sigma_1 + 3\sigma_3 > 0 \qquad (9)$$

and

$$\cos 2\theta = \frac{\sigma_1 - \sigma_3}{2(\sigma_1 + \sigma_3)} \qquad (10)$$

where:
T_s = uniaxial tensile strength of rock substance and
θ = angle between minor principal plane and plane of failure.

By combining Eqs 9 and 10, a failure equation can be obtained for comparison with Mohr's strength equation:

$$\tau_f = 2\ (T_s\sigma + T_s^2)^{1/2} \qquad (11)$$

where:
τ_f = shear stress on plane of failure and
σ = normal stress on plane of failure.

By plotting such an equation, a curve is obtained indicating a much

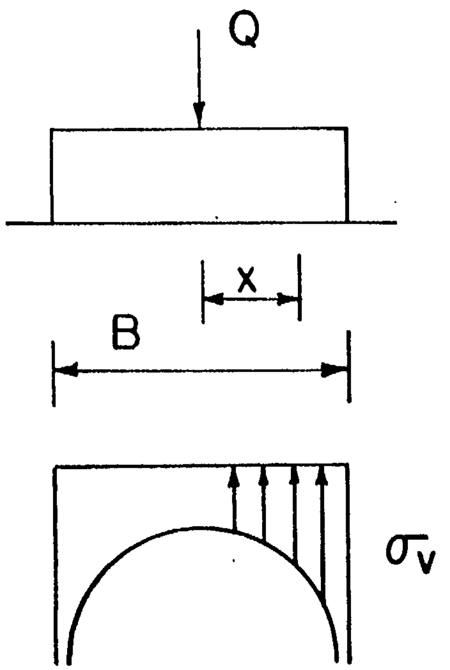

FIG. 2—*Stress concentration in brittle rock under rigid bearing.*

lower tensile strength than would be deduced from a linear envelope, such as might be extrapolated from compression tests using Mohr's strength theory. It also provides an envelope on the compression side of the origin with a decreasing slope starting at 45 deg at the *Y* axis. Both of these aspects are in general agreement with the results of triaxial testing on rocks.

If a wedge analysis, as shown in Fig. 1, is made using Griffith's strength theory, an equation can be obtained for the bearing capacity of rock. As such an equation would only be applicable to brittle and, hence, generally strong rocks, it has been assumed that the contribution to the resistance to failure of the force of gravity on the ground itself is negligible. With this assumption the simple resultant expression for a long load is

$$q_f = 24\ T_s = 3\ Q_u \qquad (12)$$

where Q_u = uniaxial compressive strength of the rock.

The end effects for a square bearing area and the effect of any surcharge, p, have not been analyzed.

The above equations are based on the assumption that the strength of the rock will be mobilized at the same time along the entire failure surface. When more is known about the failure of brittle materials, it is probable that we shall find that failure is initiated at a point due to a concentration of stress and propagates into a progressive failure. Furthermore, although experimental work on glass has substantiated Griffith's theory very well, insufficient work has been done on rocks to determine whether this theory would predict compression or bearing failures.

Rigid Bearing—For tests conducted on brittle rocks it is improbable that there will be sufficient yielding, so that average stresses along surfaces of impending failure can be used in the analysis as opposed to local stress concentrations which will cause failure of the rock at a point.

For brittle rocks under a rigid foundation the stress concentration, as shown in Fig. 2, at the edge of the bearing area could be significant. Using Eq 5, the average bearing pressure at failure would be as follows:

$$q_f = Q/B = \pi\sigma_v(y/B)^{1/2}(1 - y/B)^{1/2} \qquad (13)$$

where y = distance in from edge of foundation.

From Eq 12 we can postulate that failure will occur when the stress in the rock, σ_v, is equal to three times the uniaxial compressive strength. Also, we can assume that towards the edge of the foundation the lack of confinement together with the high stress level would cause some plastic reaction in a bearing medium, such as concrete, so that the theoretically infinite stress would not occur; consequently, the maximum stresses would be equal to that which would be calculated at some distance y in from the edge of the foundation.

Using these assumptions Eq 13 is modified:

$$q_f = 3\pi Q_u(y/B)^{1/2}(1 - y/B)^{1/2} \qquad (14)$$

This equation is based on the concept that failure will be initiated at a point under the foundation at a distance y in from the edge, and, because the foundation rock is brittle, a progressive breakdown would then occur. Conceivably this equation could have the more general form:

$$q_f = K\,Q_u/B^n \qquad (15)$$

where K and n are parameters and possibly constant for certain ranges of conditions.

The above equations for bearing capacity suggest that the effect of the width of the bearing area, B, varies with the type of material. For a yielding rock that approaches the properties of a soil with the bearing plate on a horizontal surface, Eq 6 shows that the bearing capacity will increase with the width of the bearing area. If the subgrade material can be considered as frictionless (for example, a shale with a high positive pore water pressure might behave this way) or where the bearing is occurring on a vertical surface, the bearing capacity is independent of the width of the bearing area. Also, for brittle rocks under a flexible loading or a uniform bearing pressure, the bearing capacity is likely to be independent of the width of the foundation. Then for a rigid footing on a brittle rock the bearing capacity is likely to vary inversely with the width of the foundation.

Furthermore, there is an additional case that can occur where a rigid

FIG. 3—*Plate-load test on the wall of a drift.*

footing bears on a hard rock which is overlying a softer stratum [5]. Here the stress concentrations under the edges of the footing would be even greater than represented above. In this case, it is probable that the bearing capacity would vary inversely with the width of the bearing area raised to some greater power of n as expressed by Eq 15.

The Tests

Method

These plate load tests were conducted on the walls of drifts, as shown in Fig. 3. A hydraulic prop was used to apply the load on a circular steel plate. The load was obtained from the calibrated hydraulic gage. Three $\frac{1}{1000}$-in. dial gages were used to measure the deflection of the plate.

The load was applied in increments, and the deflection was read for each increment of load.

Besides simply applying a load to the plate, certain time limitations were observed. It is important for static problems to control the rate of application to ensure that all viscous components of deformation are obtained for each increment of load. Failure in some materials can be by continuous plastic flow at a relatively slow rate. In this case, if a fast rate of load application were used, the test results would overstate the actual strength of the material. The test specifications that were used are included in the Appendix.

In addition, one or more load increments were cycled to determine the modulus of deformation from the recovery curve. This modulus is considered to be a better measure of the rock properties unaffected by any surface loosening or expansion.

TABLE 1—*Plate-load test results on iron ore.*

Test No.	Plate Diameter, in.	Bearing Pressure at Failure, psi	Cohesion,[a] psi	Moisture Content, %	Deflection at Failure, in.	Modulus of Recovery,[b] psi
11	6.5	685	7.5	12.2	0.52	20 000
12	5.0	1555	17.1	4.0	0.24	78 200
13	5.0	1580	17.4	3.9	0.40	40 300
14	5.0	1500	16.5	7.9	0.65	33 700
15	4.0	2000	22.0	4.0	0.33	46 400
16	4.0	2400	26.4	5.2	0.12	164 000
17	5.0	1500	16.5	10.2	0.19	58 500
26	5.0	924	10.1	13.7	0.32	38 300
27	5.0	689	7.6	19.7	0.75	...
28	6.5	851	9.4	12.9	0.60	41 800
29	6.5	851	9.4	12.4	0.42	42 700
30	4.0	2155	23.6	5.4	0.35	99 900
31	4.0	1806	19.9	3.9	0.60	43 400

[a] Cohesion was calculated assuming $\phi = 37$ deg, obtained from the laboratory testing of the recompacted ore.
[b] This modulus was determined from the recovery of deformation during an unloading cycle using Eq 3.

The purpose of testing was to determine the deformation and strength properties of three sedimentary rock formations of Proterozoic age: an iron ore, a so-called paint rock, and an ash rock.

Iron Ore Results

Thirteen tests were run in two drifts in iron ore. The structure of the rock was that of hard grains set in a fine-grained, usually soft matrix. It could be classified as a weak to strong, plastic, massive, blocky to solid rock [*6*]. The maximum particle size varied between ¾ and 1¾ in.

The results of these tests are shown in Table 1. The mean bearing pressure at failure was 1423 psi, and the coefficient of variation was 39.0 per cent. The mean modulus of deformation (or recovery) was 3.76 × 10^4 psi with a coefficient of variation of 63.8 per cent.

The nature of failure was not the same for all the tests. For Test 11 failure was by sudden yielding with circumferential cracks appearing on the surface of the rock about 1 in. beyond the edge of the plate. For Test 12 brittle cracking noises preceded the ultimate bearing failure. These noises started at a bearing pressure of 923 psi, and pieces of rock started to fly off the surface at pressures greater than 1000 psi with failure ultimately occurring at 1555 psi. In Tests 13, 14, 16, and 17

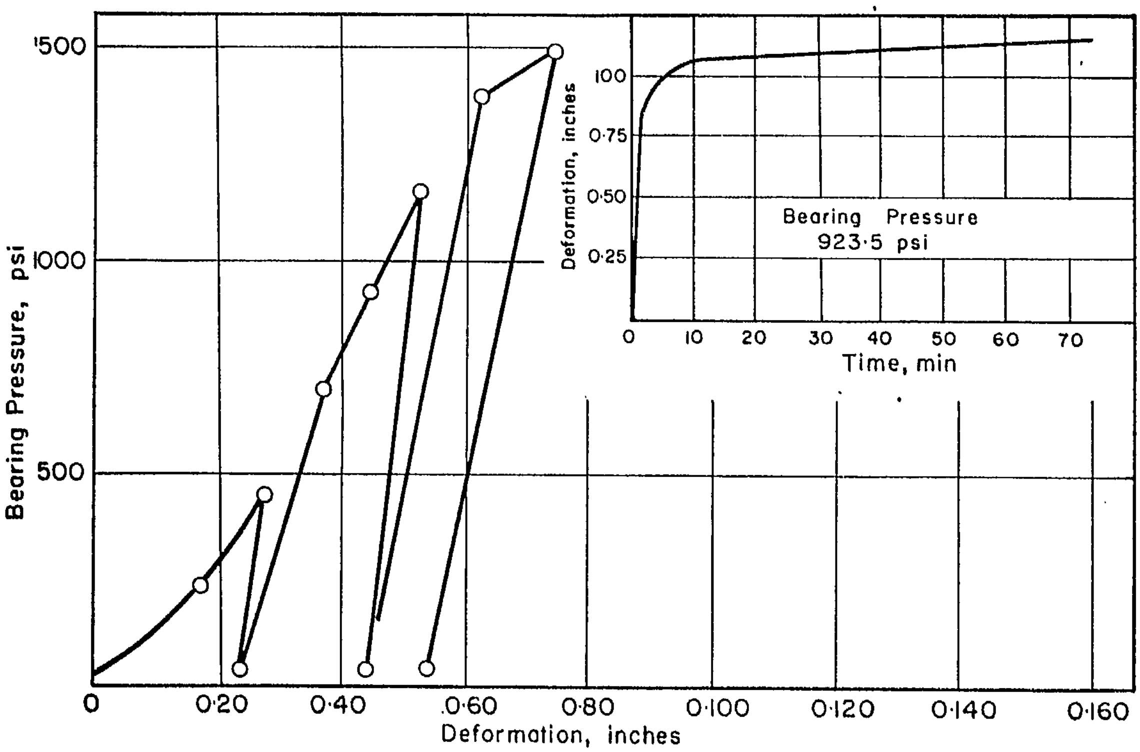

FIG. 4—*Typical bearing pressure versus deformation and deformation versus time curves for iron ore.*

failure occurred suddenly in a brittle manner without any noise preceding failure. In Test 15 failure was by plastic yielding; however, pieces of ore started flying off the face at a bearing pressure of 1889 psi. The other tests had a similar scatter of failure patterns.

Typical bearing pressure versus deformation and deformation versus time curves for the iron ore are shown in Fig. 4. These show that the rock exhibited little viscosity but produced considerable plastic, or irrecoverable, strain.

Field density tests showed that the void ratio of the ore varied between 0.16 and 0.54. Furthermore, it was obvious from experience that the strength of the ore varied considerably, which, together with the difficulty of sampling a material essentially composed of hard rock in a relatively soft matrix, militated against laboratory testing which might have been used for comparing theoretical relations between bearing pressure at failure and either uniaxial compressive strength or c/ϕ parameters.

As it was thought that the bearing failures could have been the result of yielding and shear failures, the results were interpreted, as shown in Table 1, using Terzaghi's bearing capacity theory. An angle of internal friction of 37 deg was assumed to be applicable to the ore, as this angle had

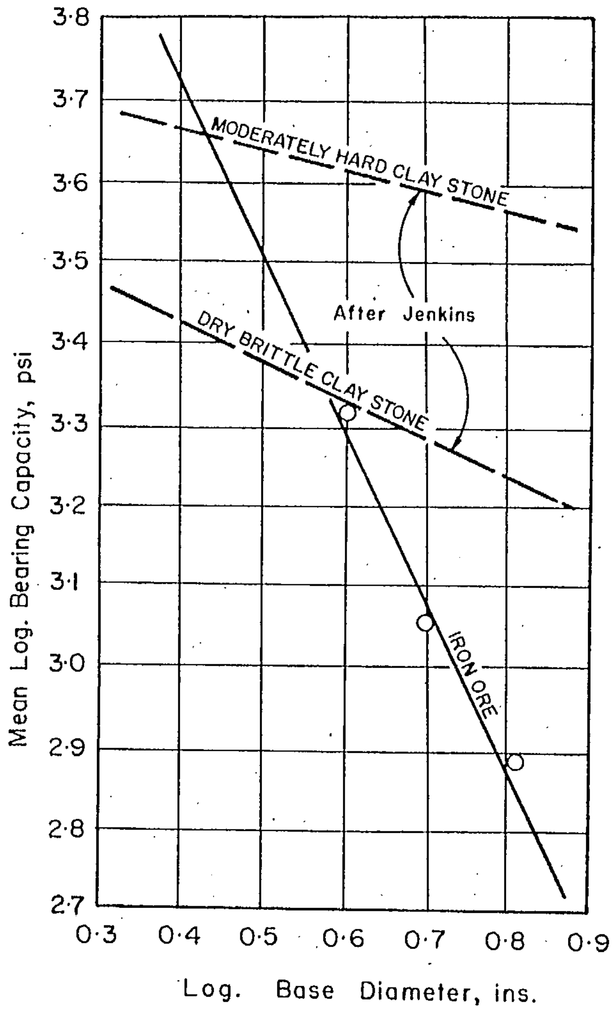

FIG. 5—*Correlation between bearing capacity and diameter of bearing plate.*

been obtained from triaxial testing on recompacted specimens at similar void ratios that were being studied for other problems.

On the other hand, there was evidence that the ground involved in the bearing failure behaved like a brittle material. By examining the test results, it can be seen that the bearing capacity has decreased with an increase in size of plate. This suggests that the material properties should be evaluated using Eq 15.

By plotting the bearing pressure at failure against the size of plate on a log-log graph and also, by assuming that the stress concentration mechanism is valid only when the size of the plate is greater than the width of two of the component blocks of the ground, then it might be logical to extrapolate the curve back to, in this case, approximately 2½ in., to obtain a measure of the uniaxial compressive strength of the rock mass.

In Fig. 5 the log-log graph of mean bearing capacity against diameter of bearing plate is shown for this rock as well as for some tests by others [*5*]. The dependence of bearing capacity on diameter suggests that brittle failure was involved in these tests. Extrapolating back to twice the average particle size gives a bearing capacity of 5300 psi, which should represent the strength of material unaffected by stress concentrations. Using Eq 12, this bearing capacity would be equivalent to a uniaxial compressive strength of 1770 psi. Owing to the wide variation in test results on core specimens, no independent check on the validity of these deductions can be made. This strength seems a little high compared to calculated pillar stresses of from 450 to 675 psi, which sometimes caused failure [*7*].

Also, from Table 1 it can be seen that the modulus of deformation calculated from the recovery deflection curve varies with the ultimate bearing capacity. In other words, these tests provide some substantiation for assuming that the modulus of deformation of rock is an indirect measure of its strength.

Paint Rock Results

The so-called paint rock is a fine-grained mass of quartz, pyrolusite, and kaolin with subangular fragments of chert, hematite, and geothite. It could be classified as a weak, plastic, layered, solid rock [*6*]. The formation is generally made up of laminations less than ½ in. thick.

When first examined it was not known whether or not this material would have strength that was predominantly due to cohesion strength with little contribution from friction. Consequently, recompacted laboratory specimens were used to obtain some measure of the basic strength parameters. Drained triaxial compression tests were used for this purpose.

Field density tests showed that the void ratio of the undisturbed rock varied between 0.4 and 0.7. The laboratory specimens were compacted to different void ratios within this range.

The results of one series of tests run on saturated specimens with a void ratio of 0.56 showed the material to have an effective angle of internal friction of 36 deg. Another series of tests run on saturated specimens with a void ratio of 0.49 produced an effective angle of internal friction of 38 deg. Creep tests and tests varying the rate of strain indicated, contrary to expectations, that the rock properties were not sensitive to duration of loading.

TABLE 2—*Plate-load test results on paint rock.*

Test No.	Plate Diameter, in.	Bearing Pressure at Failure, psi	Cohesion,[a] psi	Moisture Content, %	Deflection at Failure, in.	Modulus of Recovery,[b] psi
18	8.0	560	6.15	10.6	1.06	19 700
19	8.0	93	1.02	14.3	0.86	...
20	8.0	135	1.48	27.1	0.70	...
21	8.0	135	1.48	13.1	0.40	...
22	6.5	175	1.92	25.4	0.20	15 500
23	6.5	108	1.18	13.3	0.22	...
24	6.5	205	2.25	13.7	0.37	36 500
25	8.0	268	2.94	7.6	0.37	47 500

[a] Cohesion was calculated assuming ϕ = 37 deg, based on laboratory testing of the recompacted footwall paint rock.

[b] This modulus was determined from the recovery of deformation during an unloading cycle using Eq 3.

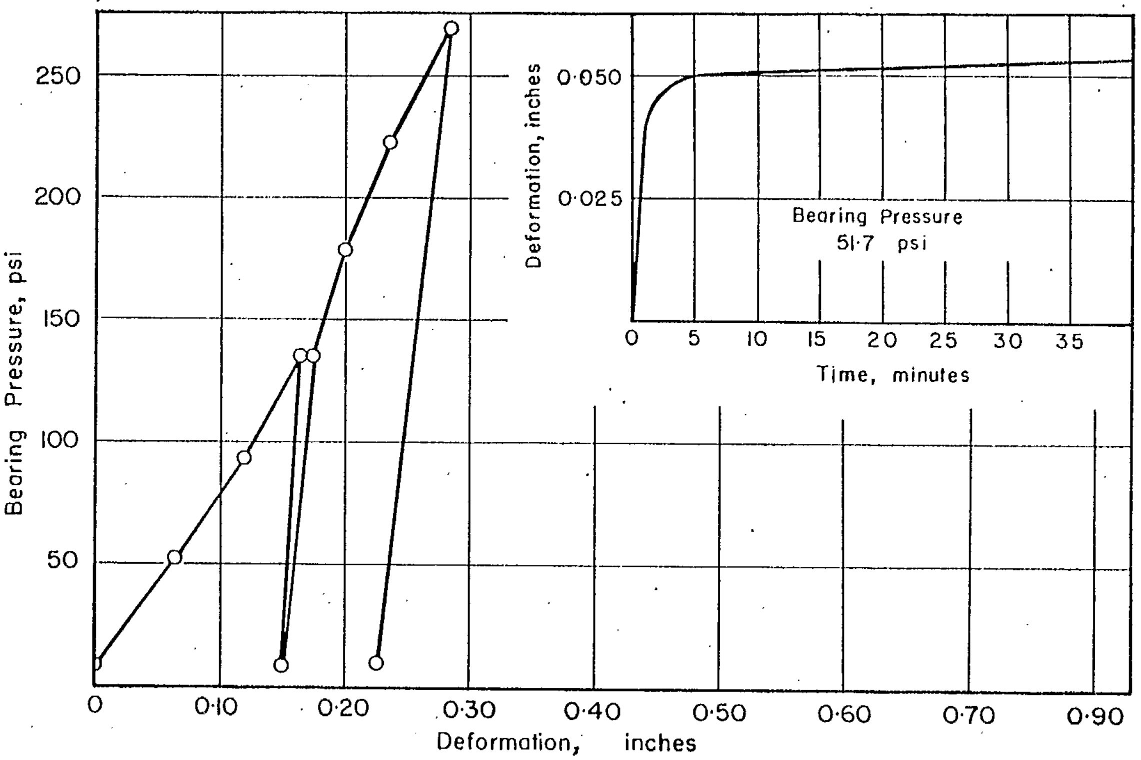

FIG. 6—*Typical bearing pressure versus deformation and deformation versus time curves for paint rock.*

In Table 2 the results of eight plate-load tests are shown. Test 19 was run on an area that contained some cracks in the face. These might have been influential in producing a low-bearing capacity at this location. From the moisture content of the material at Tests 20 and 22 and knowing the average specific gravity to be 3.00, it can be seen that in these cases the void ratio could have been greater than 0.8. Relaxation and expansion of the ground at the sides of the drifts might have occurred, which would produce higher than normal void ratios and, hence, lower than normal bearing capacities. The mean bearing pressure at failure was 210 psi with a coefficient of variation of 65.7 per cent. The mean modulus of deformation (in recovery) was 1.29×10^4 psi with a coefficient of variation of 43.3 per cent.

Typical bearing pressure versus deformation and deformation versus time curves for paint rock are shown in Fig. 6. These show that the rock exhibited little viscosity but produced considerable plastic, or irrecoverable, strain.

As there was no visual or audible evidence of brittle failure, it was assumed that a yielding failure occurred in the paint rock, which would be most appropriately interpreted using Terzaghi's bearing capacity theory for soils. Using this theory, Table 2 shows the calculated values of cohesion. These values have an average of about 1.8 psi with a maximum value of 6.2 psi. For purposes of comparison the results on recompacted

TABLE 3—*Plate-load test results on ash rock.*

Test No.	Plate Diameter, in.	Bearing Pressure at Failure, psi	Cohesion,[a] psi	Moisture Content, %	Deflection at Failure, in.	Modulus of Recovery,[b] psi
32	4.0	1070	8.3	21.3	0.52	...
33	5.0	2075	16.0	20.2	0.31	90 300
34	5.0	689	5.3	19.6	0.59	...
35	5.0	924	7.1	22.1	0.31	67 900
36	4.0	1071	8.3	21.3	0.25	112 000
37	4.0	1254	9.6	23.7	0.34	108 000

[a] Cohesion was calculated assuming $\phi = 40$ deg, which was based partially on judgment and partially on the failure planes obtained in the triaxial tests on the massive ash rock.

[b] This modulus was determined from the recovery of deformation during an unloading cycle using Eq 3.

laboratory specimens can be used. In these cases the effect of cohesion was found to vary from 6 to 13 psi. Also, from the results of analyzing slope failures in this formation [*8*], an average cohesion of 7.6 psi was obtained with a coefficient of variation of 22 per cent from nine slides.

From these tests there is a rough correlation between the modulus of deformation and the bearing pressure at failure for Tests 22, 24, and 25; however, the results of Test 18 are not consistent with this correlation.

Ash Rock Results

Six tests were conducted in a pyroclastic rock of an unusually basic type, called locally ash rock. Typical specimens of this rock contain dark green to black, lenticular, aphanatic, serpentinized fragments generally less than ½ in. in size in a greenish schistose matrix. The rock varies from a weak, slightly viscous, plastic altered material to a relatively strong, brittle schistose rock. Tests were conducted in the altered rock.

Table 3 contains the results of the plate-load tests on the ash rock. Typically, the vertical rock face at failure included several radial cracks extending out from the plate in the vertical direction, the direction of schistosity, for a distance of about 5 in. It was difficult to judge whether failure was by a brittle or yielding mechanism. However, in view of the high moisture content and, consequently, high void ratios, it is probable that a yielding failure occurred. The mean bearing pressure at failure was 1181 psi with a coefficient of variation of 36.8 per cent. The mean modulus of deformation (on recovery) was 3.45×10^4 psi with a coefficient of variation of 36.8 per cent.

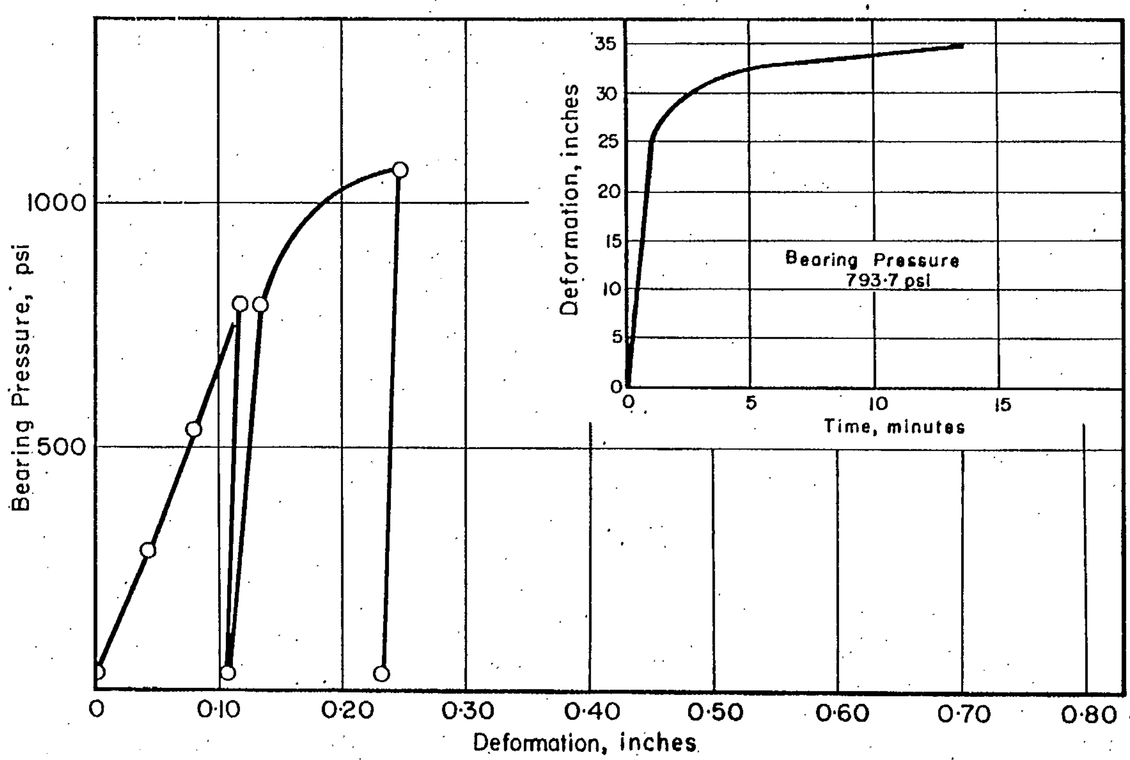

FIG. 7—*Typical bearing pressure versus deformation and deformation versus time curves for ash rock.*

In Fig. 7 bearing pressure versus deformation and deformation versus time curves are shown for ash rock. These curves show that this rock has some viscosity, or creep characteristics, and also produces considerable plastic strain.

Assuming a yielding failure, the cohesion of the rock has been calculated from the bearing pressure at failure for the various tests. The average cohesion thus obtained was 9 psi with a maximum value of 16 psi.

Although laboratory tests were conducted on core specimens of the relatively unaltered rock and also on recompacted pulverized specimens, it was not considered that the results of these tests should bear any necessary relationship to those of the plate-load tests.

Slope failures in the exposed formation were analyzed. From the results of seven slides an average cohesion of 5.3 psi was calculated with a coefficient of variation of 33 per cent. These slides were also in the altered material and could have been more affected by exposure than the material underground at the sites of the drift, thus accounting for the lower average strength. Cohesion values obtained from the two sources overlapped to a large extent.

It is interesting to compare the modulus of deformation, as obtained from the recovery curves on these plate-load tests, with a compilation that has been made in terms of an alteration index which is obtained by multiplying the void ratio by 100 and dividing by the specific gravity of the solid [*9*]. The alteration index of the ash rock, therefore, would be about 3.7. Previous tests indicate that for this alteration index a range in modulus of deformation could be expected of from 80,000 to 110,000 psi [*9*], which is remarkably close to those obtained from these tests.

Conclusions

1. Some useful corroborative information was obtained on the *in situ* strength and deformation properties of the three different rock types subjected to plate-load tests. However, the principal aspect that emerges is the disposition of strength values which occurs in testing geological materials.

2. The testing of rocks that are brittle is complicated by the importance of stress concentrations and the importance of recognising the mechanics of failure, so that the results can be properly interpreted or extrapolated through strength parameters to prototype geometry and loadings. In tests on iron ore, the combination of brittle failure and stress concentrations under the plate seemingly produced a bearing capacity that varied inversely with the diameter of the plate, contrary to what one would expect on yielding materials such as soils.

3. Except for soft rock in tunnels or drifts, plate-load testing is an expensive method for determining rock mass properties. Large forces are required for loading, and, on the ground surface, equally large reactions must be supplied similar to those used for pile-load testing.

4. It is probable that the most favorable situation for plate-load testing is when the test requires little extrapolation to the prototype case. For example, for predicting prop penetration into weak floors underground, the plate-load test has demonstrated its usefulness; similarly, it could be useful in favorable circumstances for predicting settlement of foundations.

Acknowledgment

It is a pleasure to acknowledge the important contributions to this work of J. R. Helliwell and K. L. McRorie of Steep Rock Iron Mines Ltd. and R. C. Parsons of the Rock Mechanics Laboratory in Ottawa.

APPENDIX

Suggested Test Specifications

Plate-Load Testing

1. Three to six tests should be done at each location; individual tests are to be separated by five plate diameters or approximately 3 ft center-to-center.
2. Apply a seating load of approximately 500 lb; record the magnitude of the seating load accurately; allow plate to become stationary.
3. Use load increments of one-fifth or less of the estimated ultimate capacity; record times and deformations immediately before and after load application, and immediately before and after load release.
4. Apply three load increments (25, 50, and 75 per cent of the estimated failure load), maintaining each increment constant and taking deformation readings every minute until the rate of deflection becomes equal to or less than 0.001 in./min; then noting deformations at 5-min intervals until the rate of deformation becomes equal to or less than 0.001 in./5-min; then noting deformations at 15-min intervals until the rate becomes less than 0.001 in./15-min. This specification can be modified for subsequent tests as a result of experience with a particular material.
5. After the third load increment, release the load to the seating load, and record the deflection to the same time rate specification as in Item 4. Reapply the previous load, and read deformations again to the same rate specification.
6. For special testing repeat Item 5.
7. Load to failure if possible.

Conventional Uniaxial Compression Testing

1. A suite of at least ten specimens of the same rock substance should be tested to obtain a significant mean and a measure of the dispersion of strength values.
2. Roller lap the specimens, if necessary, so that the maximum difference in diameter over the length of the specimen is less than 0.001 in. Lap the ends of the specimens on a wheel so that they are parallel within 0.001 in. A standard length-diameter ratio is 2:1, but a ratio down to a minimum of 1:1 is acceptable. After lapping, allow the specimens to dry at room temperature for at least 24 hr.
3. Measure the specimen to 0.001 in. at three points for the lengths and at three points for the diameter. Weigh specimens to the nearest 0.01 g. Measure strain either with two strain gages cemented at the midheight of the specimen and on opposite sides or with a compressometer that measures the change in length over a 1-in. gage length.
4. Apply the load at a rate of approximately 1000 psi/sec until failure occurs. Record the maximum load and the duration of the test. Describe qualitatively the type of failure as indicated by the noise produced, for example, very violent, violent, and quiet. Describe the orientation of the fractures, for example, top cone, bottom cone, longitudinal, diagonal, irregular, along with a description of the fragment size, for example, powdered, highly fragmented, quarter inch with slivers. Where possible determine the fracture angle.

Classification Uniaxial Compression Testing

1. Unless otherwise stated the specifications for conventional uniaxial compression testing apply. The rock substance can be classified with respect to strength as weak for Q_u less than 5000 psi, strong for Q_u between 5000 and 25,000 psi, and very strong for Q_u greater than 25,000 psi [*6*].

2. Apply the load in increments equal to approximately 0.25 Q_u, where Q_u is the assumed uniaxial compressive strength of the rock substance at a rate of loading of approximately 1000 psi/sec.

3. When the load has been established at the increment value, keep it constant for 30 min, and record strain readings at 0, 0.5, 1, 2, 5, 10, 20, and 30 min.

4. After maintaining the load increment for 30 min, unload the specimen. Maintain the specimen at zero stress for 10 min, recording intermediate strain.

5. Apply an increment of load equal to 0.5 Q_u to the specimen and maintain for 30 min with the same requirements for strain readings as for the first increment of load. Cycle the load to zero as for the first increment and then increase to 0.75 Q_u and subsequently to 1.0 Q_u.

6. The prefailure deformation characteristics of the rock substance are then classified as elastic if the strain rate at a stress of 50 per cent of the conventional uniaxial compressive strength is less than 2 microstrain per hr, and viscous if it is greater than this rate. The strain rate is determined by plotting the strain obtained during the 30 min period at a load of 0.5 Q_u against the logarithm of time and extrapolated to obtain the average rate applicable for the first hour [*6*].

7. The failure characteristics of the rock substance are classified as brittle if less than 25 per cent of the total strain before failure is permanent and plastic if it is more than this quantity [*6*].

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DISCUSSION

B. Ladanyi[1] *and D. Nguyen*[2].—The authors should be congratulated for their very interesting analysis of the plate-load test as a means for determining the deformation and strength properties of the rock mass. The discussers were particularly interested in the part of the paper dealing with the problem of the bearing capacity of rocks. They agree completely with the statement of the authors that "The most favorable situation for plate-load testing is when the test requires little extrapolation to the prototype case." On the other hand, if there is no model similitude between the test and the prototype, the results of the test can be used in design only if the phenomenon produced is well understood, so that general strength parameters of the tested material can be properly determined.

The determination of the strength parameters of the rock from a plate-loading test will require, therefore, a proper understanding of the mechanism of rock failure under a plate load. While the behavior of soils and yielding materials under a plate load at the surface is actually rather well known as a result of a number of experimental and theoretical investigations carried out during the last 40 years, in brittle materials the same problem up to now has been relatively little investigated. The analysis and observations presented by the authors on this subject are, therefore, particularly welcome.

As the mechanism of failure is concerned, the observations made by the authors in their tests performed on the walls of drifts are most interesting. The mode of failure observed in different tests was described as follows:

Test 11—"Sudden yielding with circumferential cracks."

Test 12—"Brittle cracking noises started at a bearing pressure af 923 psi, and pieces of rock started to fly off the surface at pressures greater than 1000 psi, with failure ultimately occurring at 1555 psi."

Tests 13 to 17—"Failure was by plastic yielding (at 2000 psi); however, pieces of ore started flying off the face at a bearing pressure of 1889 psi."

In the tests on the ash rock, "at failure several radial cracks [were observed] extending out from the plate."

From these observations, which are in agreement with those made by

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other investigators in similar tests, it can be concluded that, depending mostly on the type of rock, a plate-load test can produce: (1) an instantaneous brittle failure, (2) a brittle failure preceded by internal fracturing, or (3) a plastic failure.

In the first case it can be postulated that the failure of the rock will take place as soon as the condition for brittle failure is satisfied at a single point. The first crack will propagate immediately and lead to an instantaneous fracturing of a limited zone beneath and around the plate accompanied with eventual rockbursting.

In the second case the behavior of rock is similar to that observed during indentation of brittle materials by a shallow pyramidal indenter.[3] It seems that in this case, after starting at a point, the failure englobes quickly a hemispherical region beneath the plate. In this region the rock is fractured and has lost most of its cohesion; however, its shearing strength is still high due to internal friction and high compressive stresses. The fractured hemispherical zone transmits the pressure radially to the surrounding material, similarly as in the case of the expansion of a spherical hole under pressure. Under this radial pressure the rock surrounding the plate will be fractured showing radial cracks at the surface. The radial pressure may eventually lead to rockbursts if the rock is brittle or to a wedge failure if the rock is more plastic.

The third case of failure is typical for yielding and incompressible materials but can also be observed as an ultimate state of failure in certain brittle rocks.

It can be seen, therefore, that no single theory will be able to describe properly the complete phenomenon of failure under a plate for different types of rocks. In fact, there are actually three different theories available which may be found useful in interpreting the failure phenomenon occurring in the rock under a plate load. The theories are: (1) incipient failure theory, (2) theory of the expansion of a spherical hole under pressure, and (3) wedge theory.

The first theory has the object of giving the conditions for the beginning of failure at a single point or in a limited region. One approach to this problem has been shown by the authors for a rigid plate on a Griffith material (Eq 14). Another approach will be shown hereafter, for a flexible load both on a Griffith and on a Coulomb material.

The second theory has already been used in the interpretation of hardness test (see footnote 3) and can be useful for studying the second phase of failure in Case 2, where a fractured hemispherical zone is formed beneath the plate before a general failure.

The third theory, which is well known for a Coulomb material in soil

[3] W. F. Brace, "Behavior of Rock Salt, Limestone, and Anhydrite During Indentation," *Journal of Geophysical Research,* Vol 65, No. 6, June, 1960, pp. 1773–1788.

mechanics and which has been applied to a Griffith material by the authors (Eq 12), is able to describe successfully either the complete phenomenon of failure in a yielding material or the ultimate state of failure in brittle and intermediate materials.

From the above considerations it can be concluded that the determination of strength parameters from plate-load test results may present considerable difficulties. In order to be able to obtain results close to reality, the investigator should make correct assumptions on the following problems: (1) Which failure criterion is applicable to the rocks tested? (2) To which phenomenon or to which phase of the total phenomenon of failure under a plate does the load adopted in the test as the failure load correspond? Only with this knowledge a correct theory can be chosen and the strength parameters properly determined. Finally, it may be interesting to show an alternate approach to the problem of incipient failure under a plate load.

It is known from a similar analysis in soil mechanics[4] that the condition for incipient failure under a uniformly loaded strip can be determined from stress distribution in an elastic half space according to Boussinesq. In soils, owing to their yielding character, the approach has not been very successful. However, for strong and brittle rocks, whose elastic properties remain practically unchanged up to failure, this approach may be of more interest.

The concept consists simply in finding the minimum load at which the failure condition is satisfied in the ground, at least in a single point.

As in the case of a plate-load test the influence of gravity forces on the stress distribution is negligible, the problem becomes very simplified. If, for example, a uniformly loaded strip is assumed, the principal stresses are given by

$$\left.\begin{aligned}\sigma_1 &= \frac{q}{\pi}\,(\psi_0 + \sin\psi_0)\\ \sigma_3 &= \frac{q}{\pi}\,(\psi_0 - \sin\psi_0)\end{aligned}\right\} \qquad (16)$$

where q is the uniform pressure applied on the strip, and ψ_0 is the angle between two straight lines drawn from the considered point across both ends of the strip.

Substituting the stresses according to Eq 16 in the original Griffith's equation (Eq 10) and differentiating with respect to ψ_0, it is found that the lowest value of the load for which the failure condition is satisfied, at least in one point, is

$$q_f = 2.18\, Q_u \qquad (17)$$

where Q_u is the uniaxial compressive strength.

[4] O. K. Fröhlich. *Druckverteilung im Baugrunde*, J. Springer, Berlin, 1934.

If, on the other hand, it is assumed that the modified Griffith failure theory is valid, which in the most part of compression region does not differ from the Coulomb theory, it is found from a similar analysis that the value of q_f for incipient failure is not a constant but is a function of the slope angle ϕ of the failure envelope, as expected.

The ratio q_f/Q_u is found to vary with ϕ from 2.30 at $\phi = 30$ deg, over 2.74 at $\phi = 40$ deg, to 3.43 at $\phi = 50$ deg.

A similar analysis can be made for a uniform circular loading; however, in this case the analysis was limited to the points located on the vertical axis only for which the following simple expressions for principal stresses are valid:

$$\left.\begin{aligned} \sigma_1 &= q\left(1 - \cos^3\frac{\psi_0}{2}\right) \\ \sigma_3 &= \frac{q}{2}\left(2 - 3\cos\frac{\psi_0}{2} + \cos^3\frac{\psi_0}{2}\right) \end{aligned}\right\} \qquad (18)$$

where ψ_0, as before, denotes the angle between two straight lines drawn from the considered point across two diametrically opposite points on the edge of the loaded circular surface.

Following the same procedure as before it is found that, in the vertical axis, the failure will be initiated, when

$$q_f = 2.715\, Q_u \qquad (19)$$

if the original Griffith theory is assumed. For the modified Griffith theory, on the other hand, it is found that the ratio q_f/Q_u, corresponding to incipient failure, will have values of 3.12 at $\phi = 30$ deg, 4.09 at $\phi = 40$ deg, and 6.25 at $\phi = 50$ deg.

It is interesting to compare the above values of incipient failure loads with those corresponding to more advanced phases of failure.

For a Griffith material the above incipient load, $q_f = 2.18\, Q_u$, should be compared with the load $q_f = 3Q_u$ (Eq 12) obtained by the authors for a wedge failure. (In fact, a slightly greater value of the load may be expected in the last case when a kinematically admissible solution will be found.) For a circular loading the greater incipient failure load of 2.715 Q_u suggests that a wedge failure may be attained at about 4 Q_u.

For a modified Griffith (or Coulomb) material the ultimate failure load for a strip can be calculated by using Prandtl's theory. The values found for the ratio q_f/Q_u in this case are as follows:

ϕ	= 30 deg	40 deg	50 deg
q_f/Q_u	= 8.7	17.6	48.6

The values should be compared with those obtained above for the incipient failure in the same material. It will be seen that in such a material there is a large difference between the incipient and the ultimate failure loads, respectively. The region of loading between the incipient and ultimate failure is thought to correspond to the intermediate phase of failure during which the local fractured zone beneath the plate is acting on the surrounding material as an expanding spherical bulb.

D. F. Coates and M. Gyenge (authors)—The discussion that has been submitted by B. Ladanyi and D. Nguyen is a very good review of the possible mechanisms of rock failure under bearing pressure. Even though their theoretical comparisons are made with the assumption of a uniform bearing pressure, the differences between the bearing pressures to produce the start of failure as opposed to a general and complete failure is instructive. However, for the majority of cases of bearing pressure on rock, significantly nonuniform loading probably exists; in other words, the structure applying the load has some rigidity. Consequently, it would be interesting to have similar analyses made taking into account the relative stiffness or rigidity of the structure applying the pressure with respect to the foundation material.