Bearing Capacity of Flexible Pavements Subject to Frost Action

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The bearing capacity of flexible pavements is reduced considerably during the spring break-up period partly by reduction in relative density of the subgrade material as a result of frost action, partly by the saturation of the soil caused by thawing and partly by excess pore pressure resulting from the incomplete reconsolidation of the subgrade.

A method has been developed by which the ultimate bearing capacity of flexible pavements can be evaluated. This method is based on the assumption that the subgrade soils are fully saturated and that the wheel loads causing failure are applied so rapidly that no change in water content takes place during loading.

The calculated ultimate bearing capacity has been compared to that determined by a method proposed by Linell. Good agreement was found between the two methods.

•THE BEARING CAPACITY of flexible highway or airfield pavements is reduced considerably during the spring break-up period. Consequently it is often necessary to reduce drastically the allowable wheel or axle loads during this period in order to protect these pavements from damage. These restrictions on highway traffic often cause considerable economic hardship to the users. The present methods (1, 2) of estimating the maximum wheel or axle loads that can safely be allowed on pavements are based mainly on experience gained from observations of the behavior of similar pavements constructed on similar types of soils and subjected to similar loading conditions. The collection of field data and their interpretation are extremely tedious and difficult, but necessary.

Furthermore, the empirical methods presently available can only be used for conditions similar to those from which the methods were derived. It is difficult to calculate accurately the bearing capacity of new types of pavements constructed outside the area where these empirical methods were developed or supported by soils with unusual characteristics and subjected to unusual types of loading. When the empirical must be extended, if the actual bearing capacity is grossly overestimated, then the allowable wheel or axle loads could cause extensive damage to the pavements. On the other hand, if the actual bearing capacity is grossly underestimated, then the corresponding restrictions could be unduly severe.

ASSUMPTIONS

The hypothesis presented in this paper, which predicts the bearing capacity of flexible pavements during the spring break-up period, is based on the assumptions that failure of the pavement is caused by the failure of subgrade, that the subgrade is saturated, and that the load causing failure is applied so rapidly that no changes in water content of the soil take place during the load application. The proposed analysis is therefore not applicable to the case when failure takes place within the base or within the wearing courses.

During the spring break-up period, the ground-water table is often located close to the ground surface and it is believed that the condition of complete saturation is, as a rule, fulfilled. If, however, the subgrade is only partially saturated, the proposed analysis underestimates the actual bearing capacity of the soils and yields conservative results.

The water content of most soils does not change appreciably if the load causing failure is applied rapidly. The loading rate necessary to prevent changes in water content depends on the permeability and compressibility of the soil, viscosity of the pore water, and the boundary conditions. The permeability of the soil is the most important of these factors. If the subgrade consists of clean sand the permeability of the soil is, in general, so high that large changes in water content of the soil can take place even if the load is applied very rapidly. In this case, the proposed analysis underestimates the actual bearing capacity of the soil and the actual bearing capacity could be several times the calculated one.

Subgrades composed of clean sand are not, as a rule, subjected to frost action and no reduction in bearing capacity from this course generally takes place during the spring break-up period. The normal increases in the elevation of the ground-water table during the spring would, however, affect the bearing capacity as it affects the effective pressures in the subgrade.

The permeability of most subgrade soils is low; a small percentage of silt or clay size particles reduces their permeability greatly. Under these conditions even a load applied relatively slowly will only cause small changes of the water content of the soil. Thus it is believed that the rate of loading as caused by heavy vehicles moving at about 30 mph does not cause appreciable changes in the water content of the soil. If, however, these changes are large, the proposed analysis underestimates the bearing capacity of the subgrade and yields conservative results.

In general, the assumptions have been chosen to give a lower limit of the bearing capacity of pavements.

DISTRIBUTION OF PORE AND EFFECTIVE PRESSURES BELOW PAVEMENTS

The total pressure within a mass of soil is in general divided into effective pressure and pore pressure. The effective pressure is transmitted through the solid phase, whereas the pore pressure is transmitted through the liquid phase of the soil.

Figure 1 shows a cross-section through a road with a total thickness of wearing and base course equal to a. At a depth of $h_1 + h_2$ below the surface of the road the total vertical pressure, the sum of the effective pressure and the pore pressure, is equal to p_1. The total vertical pressure at any depth is equal to the weight of the overlying material, then

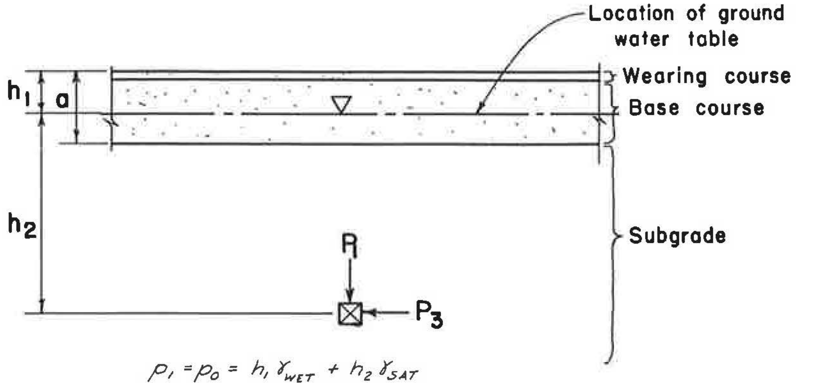
$$p_1 = h_1 \gamma_{wet} + h_2 \gamma_{sat} \tag{1}$$

in which γ_{wet} and γ_{sat} are the average wet and saturated unit weights of the base and the subgrade material, respectively. The wet unit weight is equal to the unit weight of the partially saturated material. The lateral total pressure p_2 is in general smaller than the corresponding vertical total pressure p_1 (3, 4, 5). Its value depends on the properties of the soil, method of compaction, and the subsequent stress-history of the soil. Relatively little is known of the effects of the lateral pressures on the pore pressures caused by sudden application of load (6).

The initial pore pressure u_0 is governed by the location of the ground-water table as shown in Figure 1. The ground-water table is assumed to be located within the base course at a depth h_1 below the ground surface. The pore pressure u_0 can be computed as

$$u_0 = h_2 \gamma_w \tag{2}$$

The vertical effective pressure $\bar{p}_1$ is equal to the difference between the total pressure



$$p_1 = p_0 = h_1 \gamma_{WET} + h_2 \gamma_{SAT}$$

$$p_3 \cong p_0 = h_1 \gamma_{WET} + h_2 \gamma_{SAT}$$

$$u = h_2 \gamma_W$$

$$\bar{p}_1 = \bar{p}_0 = h_1 \gamma_{WET} + h_2 \gamma_{SUB}$$

$$\bar{p}_3 \cong \bar{p}_0 = h_1 \gamma_{WET} + h_2 \gamma_{SUB}$$

γ_{WET} = Unit weight of the partially saturated pavement material.

γ_{SAT} = Saturated unit weight of base course or subgrade material.

γ_W = Unit weight of water

Figure 1. Stress distribution below flexible pavements.

and the pore pressure, then

$$\bar{p}_1 = h_1 \gamma_{wet} + h_2 \gamma_{sub} \tag{3}$$

where γ_{sub} is the submerged unit weight of the base and the subbase material. The lateral effective pressure equal to $K_0 \bar{p}_1$ depends on the coefficient of lateral earth pressure at rest K_0. The value of K_0 depends on the strength properties of the soil and on its stress history (7). (It is assumed in the following analysis that $K_0 = 1.0$ in.)

The distribution of the initial total and effective stresses in any plane can be illustrated by means of Mohr's stress circles (Fig. 2). As it is assumed that the total and effective stresses are constant in all directions, the corresponding Mohr's stress circles for total effective stresses are points located at a distance p_0 and $\bar{p}_0$ from the origin on the normal stress axis.

BEHAVIOR OF SOIL UNDER MOVING LOADS

Large excess pore pressures develop within the subgrade due to the application of moving load and the low permeability of the subgrade material. The pore pressures can be estimated by means of the pore pressure coefficients A and B if the loads are applied so rapidly that no change in water content takes place during the load application (7, 8, 9).

The wheel load causes an increase of the total all-around ambient pressure by dp_3 (Fig. 3). The portion of this pressure increase carried by pore pressure du_b is equal to the pore pressure coefficient B as expressed by

$$du_b = B dp_3 \tag{4}$$

The pore pressure coefficient B is equal to 1.0 for saturated soils (7, 10, 11). The total increase in ambient pressure is carried by an equal increase in pore pressure.

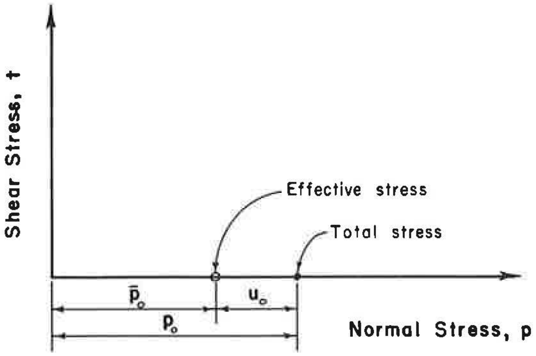


Figure 2. Mohr's stress circles for total and effective stresses.

The effective pressures in the soil are not, therefore, affected by a change of the total ambient pressures within a soil if the water content of the soil remains unchanged while the load increase is applied to the soil.

The applied load also causes an increase of the total deviator stress by $(dp_1 - dp_3)$. This deviator stress causes an increase of the pore pressure by du_a. The share carried by an increase in pore pressure for a saturated soil is equal to the pore pressure coefficient A, as expressed by

$$du_a = A\,(dp_1 - dp_3) \tag{5}$$

Experiments have indicated that the pore pressure coefficient A at failure for a sandy or silty soil is mainly a function of the relative density of the soil. At failure, the coefficient A_f could be as low as -0.32 for a dense sand and as high as 0.08 for a loose sand (7, 11).

The total increase in pore pressure caused by a sudden increase in ambient and deviator pressure is $du_a + du_b$ (Fig. 3).

The distribution of total and effective stresses in any plane can be expressed at failure by means of Mohr's stress circles (Fig. 4). Failure of the soil takes place when the shear stress developed within a body of soil exceeds its shearing strength.

The shearing strength of a soil is governed by

$$s = c_s + \bar{p}_f \tan \phi_s \tag{6}$$

in which c_s and ϕ_s are the apparent cohesion and apparent angle of internal friction of the soils as measured by the drained direct shear or triaxial test and $\bar{p}_f$ the effective normal pressure on the failure plane (13). The apparent cohesive strength c_s is equal to zero for sands, most silts, and normally consolidated clays (7). The apparent cohesive strength is larger than zero for compacted and for overconsolidated clays.

Failure of the soil takes place when the effective Mohr's stress circle is tangent to the envelop curve with a slope equal to ϕ_s. At the point of tangency the shear stress developed within the soil is equal to the shearing strength of the soil. The difference between the maximum and minimum principal stresses $(p_1 - p_3)_f$ at failure can be expressed in terms of the initial effective pressure p_o, the apparent angle of internal friction ϕ_s and the coefficient of lateral earth pressure at rest k_o, and the pore pressure coefficient A_f (7, 14) as follows:

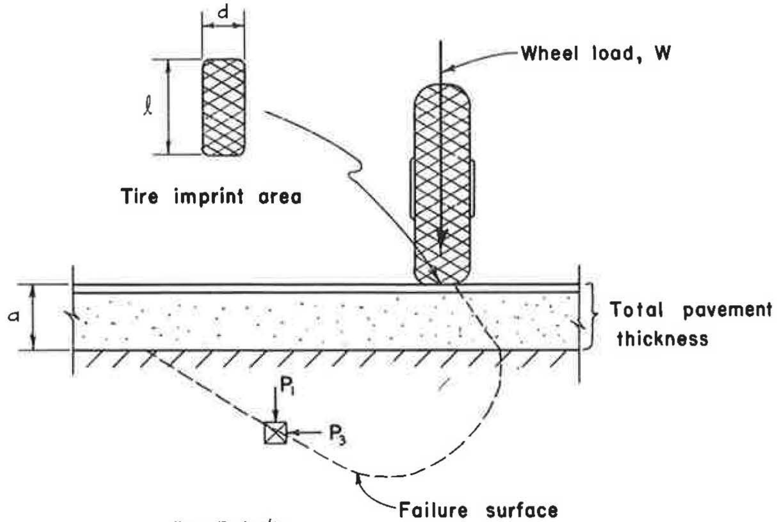
$$(p_1 - p_3)_f = \frac{2\bar{p}_o \sin \phi_s \,[K_o + A_f\,(1-K_o)]}{1 + (2A_f - 1)\sin \phi_s} \tag{7}$$

This equation is derived on the assumption that c_s is equal to zero. The principal stress difference $(p_1 - p_3)_f$ depends only on the initial stress conditions ($\bar{p}_o$, k_o) and on properties of the soil (ϕ_s, A_f).

A saturated soil with low permeability and subjected to high rates of loading behaves as a frictionless material (7) with an undrained cohesive strength equal to the radius of the Mohr's stress circle as expressed by

$$c_u = \tfrac{1}{2}\,(p_1 - p_3)_f \tag{8}$$

A loose saturated sand or silt with an angle of internal friction ϕ_s equal to 30° and a value of the pore pressure coefficient A_f and of the coefficient of lateral earth pressure at rest K_o of 0.08 and 0.5, respectively, has an apparent cohesive strength of 0.47 $\bar{p}_o$. The effective initial vertical pressure depends on the depth below the ground surface, the unit weight of the base course and the subbase material, and on the location of the ground-water table. For example, the apparent undrained cohesive strength C_u of loose saturated sand or silt is 88 psf at a depth of 2.0 ft below the ground surface if



$$p_1 = p_0 + dp_1$$
$$p_3 = p_0 + dp_3$$
$$u = u_0 + du_b + du_a$$
$$\bar{p}_1 = \bar{p}_0 + dp_1 - du_b - du_a$$
$$\bar{p}_3 = \bar{p}_0 + dp_3 - du_b - du_a$$
$$\text{Where} \quad du_b = B\, dp_3$$
$$du_a = A\,(dp_1 - dp_3) = A\,(p_1 - p_3)_f$$

Figure 3. Stress increase caused by wheel loads.

the wet and saturated unit weight of the base course and subbase material are 120 and 130 psf, respectively, if the ground-water level is located at a depth of 1.0 ft below the ground surface, and if the thickness of the base course is also 1.0 ft.

The corresponding apparent cohesive strength of a dense sand or silt is 1,280 psf at the same depth with an angle of internal friction of 34° and a value of A_f and K_o equal to - 0.32 and 1.0, respectively.

The apparent cohesive strength of a soil in its dense state is therefore approximately fifteen times the undrained apparent cohesive strength of the same soil in its loose state. It can be seen that the pore pressure coefficient A_f has a large influence on the apparent cohesive strength of a soil.

EFFECTS OF FROST ACTION

It has been observed that soils containing even a small amount of fine-grained material are subjected to frost action. Horizontal bands or lenses of pure ice with variable thickness (15, 16) are formed within the frost-susceptible material and cause a volume expansion of the soil. These ice bands have a tendency to push apart the individual soil particle. When these ice lenses melt during the spring break-up period, the soil reconsolidates under the weight of the base and the wearing course. Static compression, however, is not very effective in compacting coarse-grained soils and the density of the reconsolidated material will be low. It is believed that frost action changes the relative density of the subgrade material from dense to loose, with a corresponding change of the apparent angle of internal friction and of the pore pressure coefficient A_f. The apparent cohesive strength of the subgrade is reduced to a fraction of its initial value during the spring break-up period with a corresponding reduction of its bearing capacity.

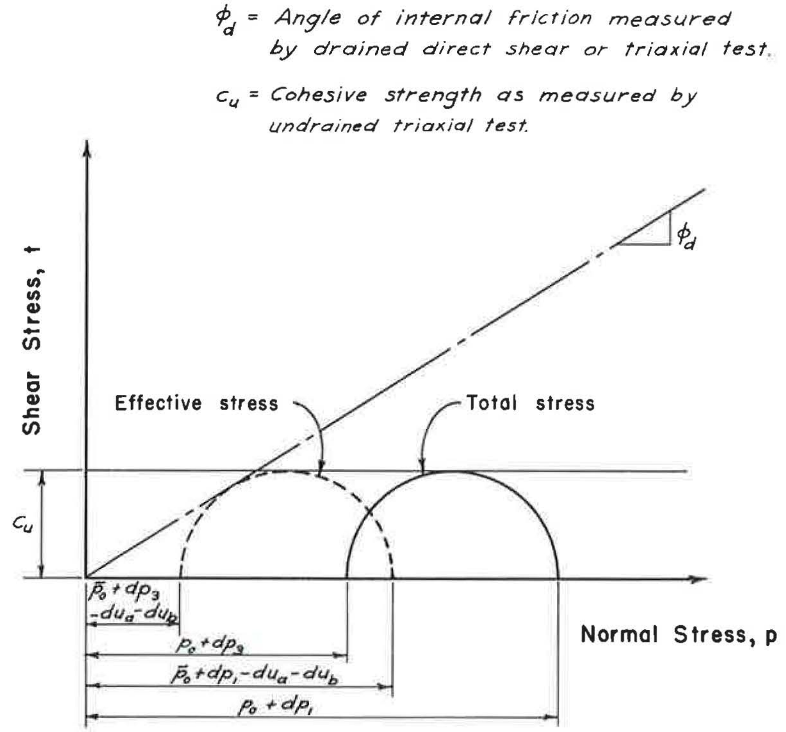


Figure 4. Mohr's stress circles at failure of the subgrade material.

Vibrations due to the moving traffic probably recompact the material in the subbase very effectively and after a few days of heavy traffic the initial density of the soil is restored.

The author has not found any information concerning the effects of frost action on the relative density and on the deformation characteristics of a soil. Further research is needed within this area before the behavior of soil under high rates of loading can be predicted accurately.

DISTRIBUTION OF STRESSES BELOW PAVEMENTS

The distribution of stresses within a mass of soil is in general calculated by means of the theory of elasticity. However, the intensity of the applied loads is so high at failure that the soil does not even approximately behave like an ideal elastic material, and elastic methods are not applicable. The distribution of load has, therefore, in this analysis been calculated by means of the 2:1 method (17, 18). This method assumes that the applied load at any depth is distributed over an area that increases in proportion to the depth below the point of loading, as shown in Figure 5.

It is assumed that at the depth a the total vertical pressure increase dp caused by the moving load W is equal to

$$dp = \frac{W}{(d + a)(\ell + a)} \tag{9}$$

It is assumed that this pressure increase is uniformly distributed over the area $(d + a)(\ell + a)$. The pressure increase dp is approximately equal to the increase calculated by means of theory of elasticity below the center of the loaded area. The actual distribution of pressure is bell-shaped and the intensity of the load at the edge of the loaded area is smaller than the calculated value. Thus the actual bearing capacity of the soil

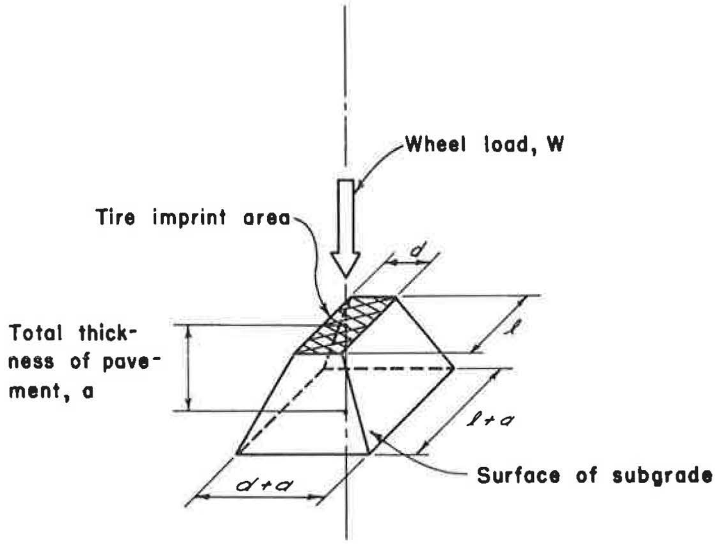


Figure 5. Assumed distribution of load at failure.

might be larger than computed, due to the approximations involved in the 2:1 method. The distribution of stresses within layered systems is not very well understood, particularly when the soil is close to failure.

ULTIMATE BEARING CAPACITY

The net ultimate bearing capacity q_{ult} of soil that acts as if it has an angle of internal friction of 0° can be calculated from

$$q_{ult} = c_u N_c \tag{10}$$

in which c_u is the undrained apparent cohesive strength of the subgrade material and N_c a bearing capacity factor, which depends on shape and size of the loaded area as well as the depth below the ground surface (19, 20, 21, 22). The net ultimate bearing capacity is equal to the ultimate capacity of the soil less the overburden pressure. The bearing capacity factor N_c is approximately equal to 7.5 for a circular or square shape of the loaded area, assuming that the thickness of the base course is approximately twice the size of the loaded area.

Failure of the soil takes place along a cylindrical failure surface extending to a depth of approximately 1.5 times the width of the loaded area. As the width of the loaded area at the depth a is equal to (a + d), the failure surface extends approximately to a depth of [a + 1.5 (a + d)] below the ground surface. The ultimate bearing capacity depends on the average shearing strength of the soil within this zone. The shearing strength of the soil is proportional to the initial effective pressure. Because the failure zone extends between the depth a to [a + 1.5 (a + d)], then the average shearing strength corresponds to the shearing strength at a depth of [a + 0.75 (a + d)] below the ground surface. The total bearing capacity of the pavement can then be evaluated by substituting Eqs. 7 and 8 into Eq. 10.

The ultimate capacity of flexible pavements subjected to frost action has been computed and shown in Figures 6 through 9. Calculations have been made for five different soils. The properties of the five soils are given in Table 1.

Soil 1 corresponds to a silt with a percentage of fines in excess of 15 percent. This material would probably be classified as type F4 material according to the Army Uniform Classification System (2) or as a ML or a MH material according to the Unified Soil Classification System (23). Soil 2 corresponds approximately to a silty sand with a content of fines in excess of 15 percent, Soil 3 to a silty gravel with a content of fines

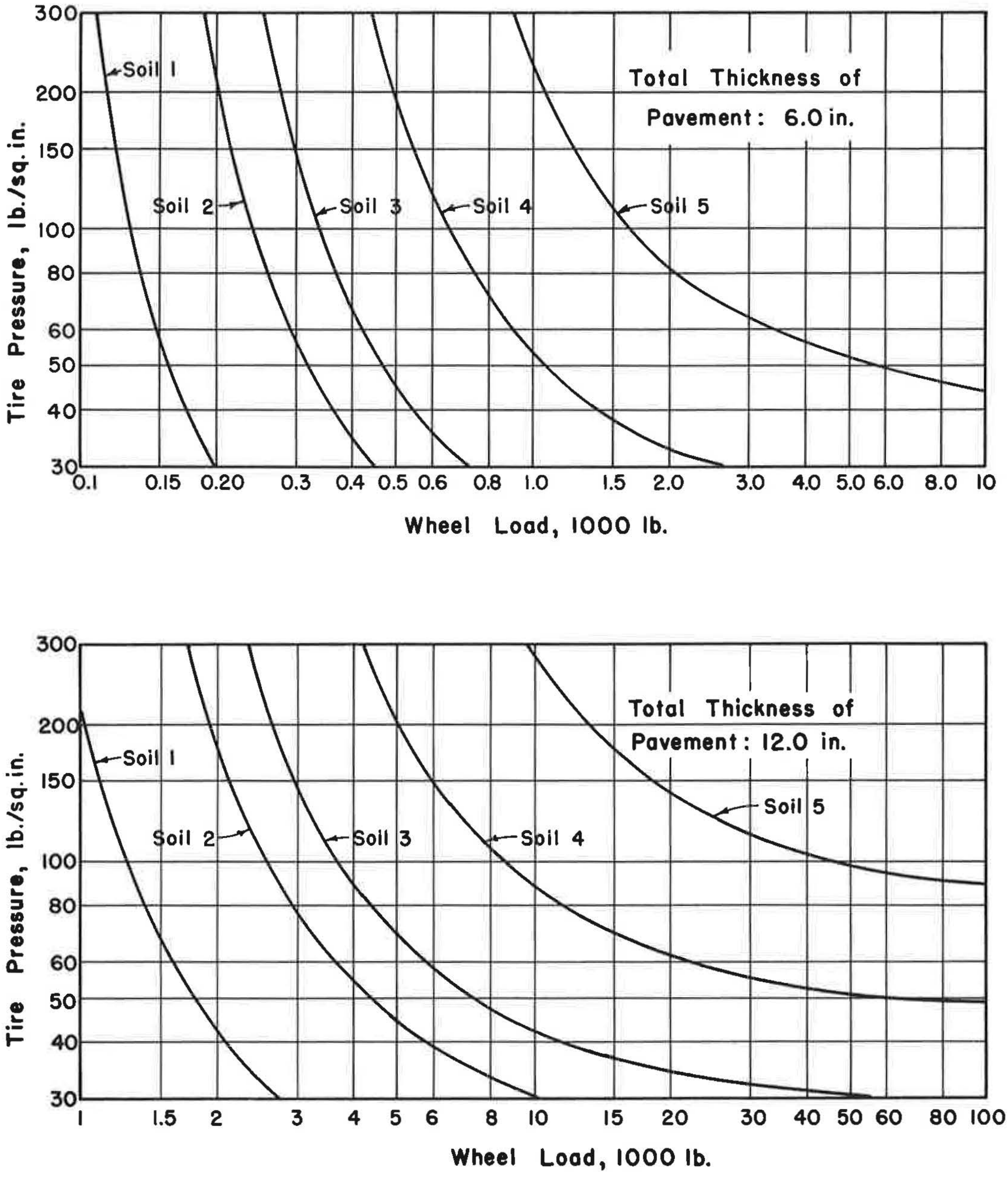
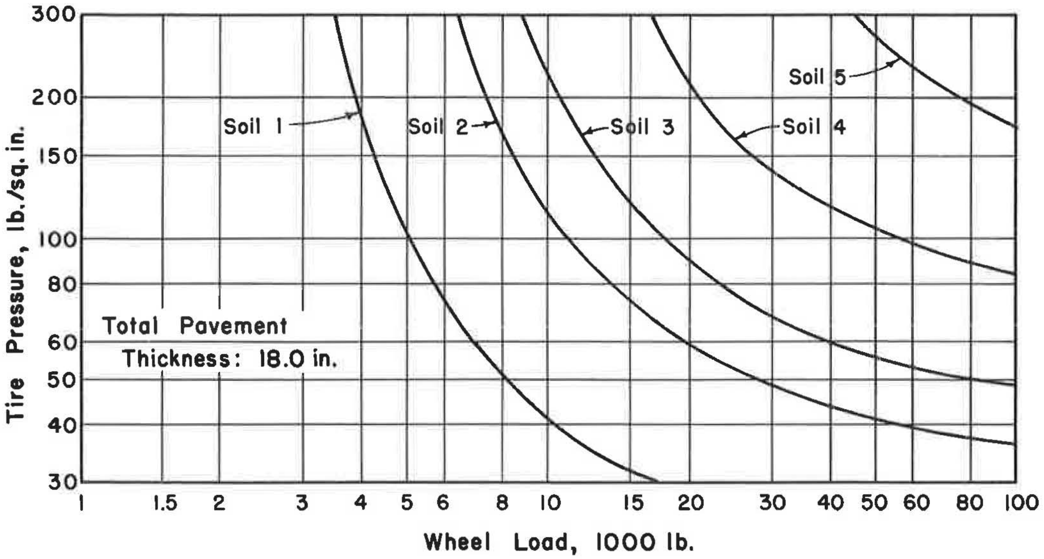


Figure 6. Ultimate bearing capacity of flexible pavements.

in excess of 15 percent. Soil 4 and Soil 5 correspond to a sand and a gravelly sand, respectively, with a percentage of fines less than 15 percent. In all calculations it has been assumed that the value of the coefficient of lateral earth pressure at rest is equal to 1.0.

The ultimate bearing capacity has been calculated under the assumption that the ground-water table is located at the surface of the subsoil at a depth a below the pavement surface. It has been assumed, furthermore, that the unit weights of the soil in the base course and the subbase are equal to 135 pcf and 125 pcf, respectively. The



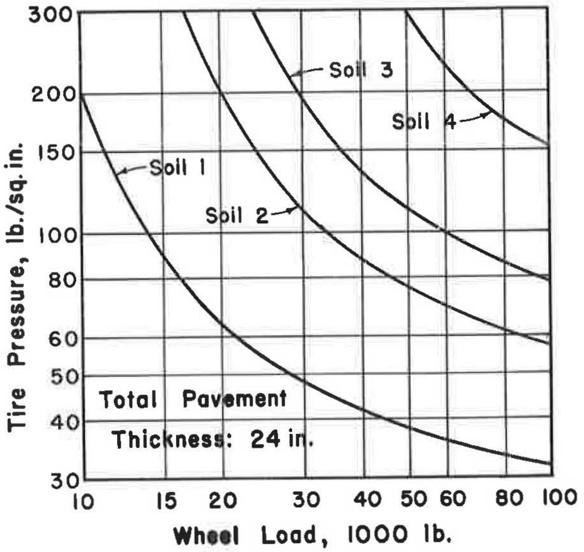


Figure 7. Ultimate bearing capacity of flexible pavements.

unit weight of water has been taken as 62.5 pcf. If the ground-water table is located within the base course, the bearing capacity is less than shown in Figures 6 through 9. It has been assumed further that the imprint area is either circular or square.

Figures 6 and 7 show the ultimate bearing capacity as a function of the contact pressure between the moving load and the pavement. The ultimate bearing capacity has been calculated for a total pavement thickness of 6, 12, 18, and 24 in. It can be seen that the total bearing capacity increases rapidly with decreasing contact pressures. This effect is especially pronounced for Soils 4 and 5.

Figures 8 and 9 show the ultimate load as a function of the thickness of the pavements calculated for the contact pressures of 100 and 300 psi, respectively. The ultimate capacity has been calculated for a total pavement thickness of 9 to 30 in.

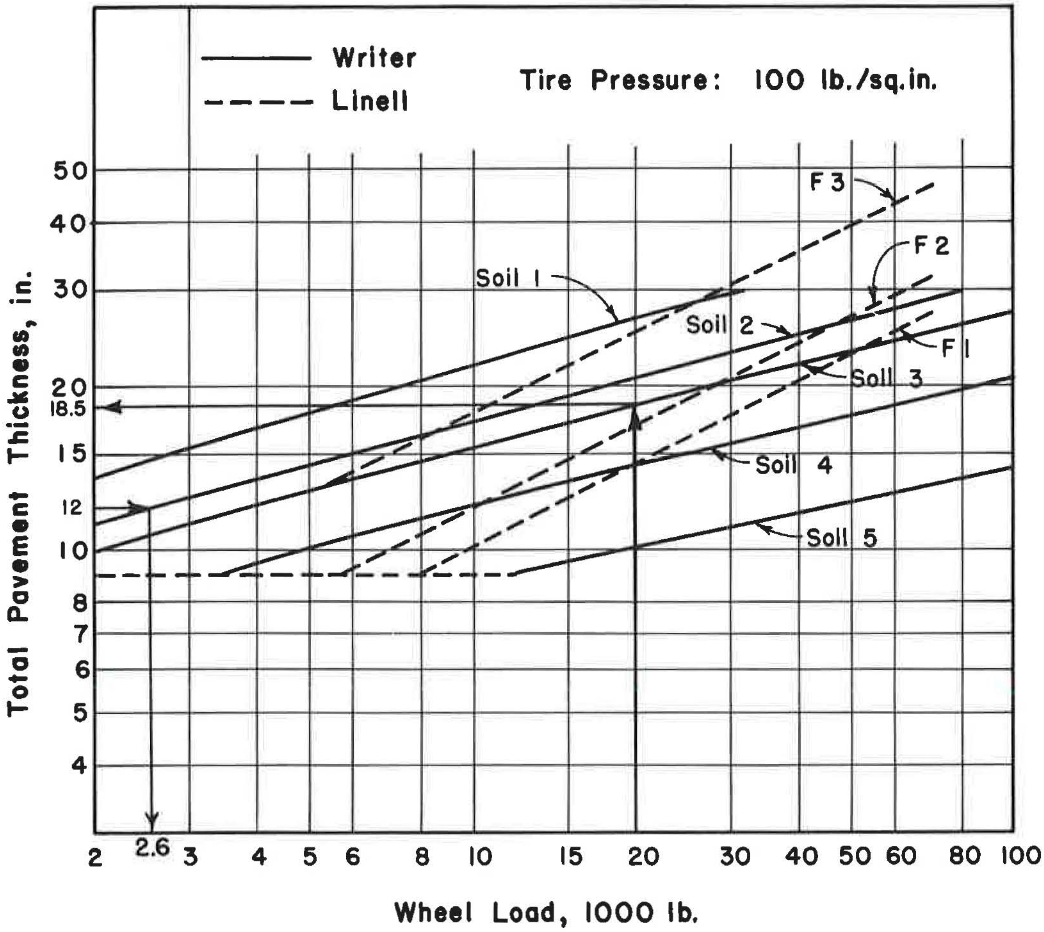


Figure 8. Ultimate bearing capacity of flexible pavements.

Figure 8 also shows the relationship between total thickness of the pavement and total capacity as proposed by Linell (2). It can be seen that the required thickness calculated by the method proposed by the author and by Linell yield approximately the same results. This general agreement indicates that the method proposed by the author yields reasonable results. However, the predicted combined thickness as calculated by the author's method is somewhat smaller for heavy wheel loads than that predicted by the method proposed by Linell. The opposite is true for low wheel loads. However, Linell's method was derived from observations of the behavior of flexible airfield pavements subjected to high wheel loads.

The relationships shown in Figures 6 through 9 are computed under the assumptions that the apparent angle of internal friction of the subgrade soil and the corresponding pore pressure coefficient A_f are known. Little is known about the value of the coefficient A_f for soils subjected to frost action. Comparisons between Soils 2 and 4 and between Soils 3 and 5 indicate that the coefficient A_f has a large effect on the bearing capacity of the soil. Therefore, it is possible that the relationships as shown in Figures 6 through 9 might change considerably as further knowledge is gained. However, the computed relationships might serve as an indication of the behavior of soils subjected to frost action.

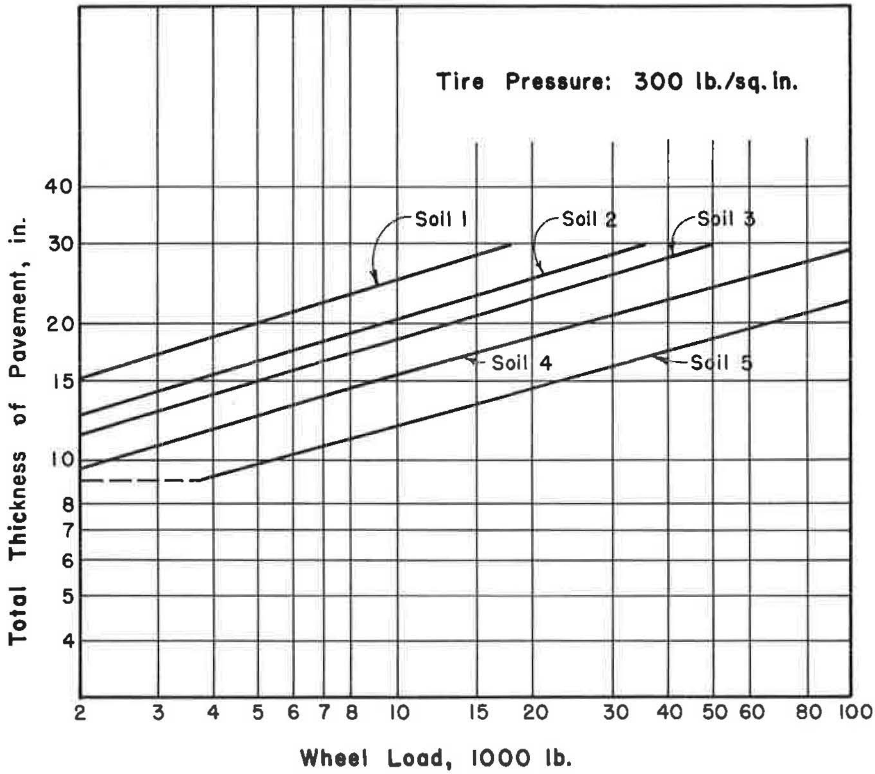


Figure 9. Ultimate bearing capacity of flexible pavements.

TABLE 1

ASSUMED VALUES OF APPARENT ANGLE OF INTERNAL FRICTION[1] ϕ_s AND OF PORE PRESSURE COEFFICIENT A

Soil	ϕ_s (deg.)	A_f	Soil Classification: Unified (23)	Soil Classification: Army Uniform (2)
1	25	0.3	ML or MH	F4
2	30	0.1	SM or SC	F4
3	35	0.1	GM or GC	F3
4	30	-0.2	SW or SP	F2
5	35	-0.2	GW or GP	F1

[1]As measured by slow direct shear or triaxial test.

DESIGN AND LOAD RESTRICTIONS ON FLEXIBLE PAVEMENTS

Figures 8 and 9 might serve as a guide for the design of flexible pavements. These two figures show the ultimate bearing capacity of the pavement as a function of its thickness. If the indicated loads are placed on the pavements, failure will take place. To prevent damage, only a fraction of the ultimate load can be allowed. The reduction of the load depends on the number of load applications during the spring break-up period. It is estimated that the allowable load should be approximately one-half its ultimate value in order to prevent damage of the pavement if the pavement is only subjected to a few load applications, and one-third of its ultimate capacity if the pavement is subjected to a large number of load applications. For example, if the maximum wheel load is equal to 10,000 lb and if the pavement is subjected to a few repetitions of load during the spring break-up period, the pavement should be designed for an ultimate load of 20,000 lb. For this total load, the required total thickness of the pavement is equal to 18.5 in. if the subsoil corresponds to Soil 3, and if the tire pressure is 100 psi. The corresponding required thickness according to Linell's method is 18.0 in.

Figures 8 and 9 can also be used to determine allowable wheel loads on airfield or highway pavements. If the total thickness of the wearing and base course is, for example, 12.0 in., and if the subsoil corresponds to Soil 2, then the ultimate capacity of the pavement is equal to 2,600 lb, as shown in Figure 8. With a limited amount of traffic during the spring break-up period, the allowable wheel load should be approximately 1,300 lb.

SUMMARY

A method has been presented by which the ultimate bearing capacity of pavements subjected to frost action can be calculated. It has been shown that the ultimate bearing capacity depends on the size and shape of the loaded area, the thickness of the flexible pavement, the location of the ground-water table, and the density of the subgrade material factors that are all known to influence the bearing capacity of pavements.

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Discussion

A. C. BENKELMAN, Altamonte Springs, Florida—The paper deals with an extremely important subject, one that should be given more consideration by research workers in the highway field. In the WASHO and the AASHO Road Tests a great deal more structural deterioration of the test sections occurred during the spring than during the summer and fall months. Considerable information was obtained regarding the condition of the subsurface components during these two periods. This information is of interest in connection with the subject matter of the paper.

Tables 2 and 3 summarize the data. For the embankment soil of the AASHO Road Test there was little difference in the percent saturation of the material in the spring and summer, 80.4 vs 78.8, and in both periods the material was well below complete saturation. However, there were appreciable differences in the indicated strength (CBR) and percent saturation of the base (high quality crushed stone) and of the subbase (uncrushed sandy gravel). This information strongly suggests that the weakness of the pavement in the spring (80 percent of the test sections failed in the two spring periods as against 6 percent in both summers) may have been due in large part to the adverse condition of the granular courses at this time.

The data for the WASHO Road Test (embankment soil only) show that only in areas that had failed completely or were about to fail was the percent saturation of the material greater in the spring (around 90 percent) than in the summer and fall (around 85 percent). The figures for unfailed areas that were sampled near the failed or about to fail areas actually show a lower level of saturation (around 83 percent) than in the summer and fall.

The embankment soil at both the AASHO and the WASHO Road Tests was a fine-grained material, an A-6 and an A-4-8, AASHO Classification. Complete data on their classification and physical characteristics are given in the published reports of the two tests.

TABLE 2

AASHO ROAD TEST MATERIALS, AVERAGE VALUES OF IN-PLACE TESTS

Time	Component	Density	M.C.	CBR	S (%)
Construction	Embankment	112.7	16.3	2.9	87.7
	Base	140.9	4.2	—	54.1
	Subbase	134.5	3.8	—	40.5
Spring	Embankment	113.5	14.6	4	80.4
	Base	143.8	4.3	87	62.5
	Subbase	136.5	5.5	24	63.4
Summer	Embankment	112.8	14.6	5.6	78.8
	Base	142.4	3.6	131	49.4
	Subbase	135.2	4.8	50	52.8

TABLE 3

WASHO ROAD TEST SEASONAL VARIATIONS IN CONDITION OF EMBANKMENT SOIL, AVERAGE VALUES OF IN-PLACE TESTS

Section	Construction Period 1952				November 1953				June 1954				Spring 1954											
													Failed Areas				About to Fail Areas				Unfailed Areas			
	M. C.	Density	S (%)	CBR	M. C.	Density	S (%)	CBR	M. C.	Density	S (%)	CBR	M. C.	Density	S (%)	CBR	M. C.	Density	S (%)	CBR	M. C.	Density	S (%)	CBR
All 22 in.	21	88	67	-	25	90	85	14	26	89	84	12												
All 18 in.	24	89	78	-	25	92	89	15	25	93	88	13												
All 14 in.	24	88	75	-	24	91	83	16	24	92	84	14												
All 10 in.	23	90	76	-	24	92	84	17	24	91	82	15	26	90	90	2	25	89	87	6	25	90	83	8
All 6 in.	24	89	77	-	22	94	81	25	23	92	80	18	24	96	91	4	22	98	90	10	22	95	82	14
Grand avg.	23	88	73	-	24	92	85	17	24	91	85	14												

Further, in the late 1940's and early 1950's a great amount of work was done by the northern States (New York included) in a study of the effect of frost action on the indicated ability of flexible pavements to support load. The work was sponsored by the Highway Research Board Committee on Load Supporting Capacity of Roads as Affected by Frost Action. It was found in the case of fine-grained soils that the reduction of plate load support values was consistently around 50 percent. In the case of sandy soils the reduction was considerably less. Because soils in most of the northern States vary in character from point to point, some of the States adopted the practice of reducing the permitted loads by 50 percent. Of course, load reduction on roads is generally limited to secondary roads or roads that are known to be weak.

At the present time a considerable number of States and all the Provinces of Canada are engaging in studies of the performance of flexible pavements in service and more work along this line is in prospect in connection with the application of the findings of the AASHO Road Test in practice. All factors (such as the condition and design of the pavement, materials, subgrade character, and traffic) are taken into consideration in these studies. Many of the States and all the Canadian Provinces are, in addition, running seasonal deflection tests of selected sections of the pavement, using the Benkelman beam. In Oklahoma and South Dakota and at the AASHO Road Test, good correlations were found between the results of beam and plate load tests. Of course, if the beam tests are a measure of the ability of a pavement to support load, a simple method of test is available for use in establishing limitation of load in the spring. In both the WASHO and AASHO Road Tests it was found that beam deflections were a fairly good predictor of pavement performance.

BENGT B. BROMS, Closure—Mr. Benkelman's discussion brought out that the degree of saturation for areas where failures have taken place at the WASHO Road Tests was higher than for areas where failures have not taken place. The degree of saturation for the failed sections was approximately 90 percent as compared with 83 percent for the unfailed sections. These data indicate that the degree of saturation is an important parameter affecting the bearing capacity of flexible pavements.

In the method of calculation proposed by the author it has been assumed that the degree of saturation of the subgrade material is 100 percent and the data supplied by Mr. Benkelman suggest that the degree of saturation of the subgrade material may approach this value under unfavorable conditions. It is possible that the degree of saturation of the soil at the time of failure might have been higher than the measured values due to drainage of the soil between the time of failure and the time of sampling and that some small changes of the degree of saturation might have taken place during sampling and testing of the soil. The bearing capacity as calculated by the method proposed by the author should not be considered as the only bearing capacity of flexible pavements but as the lower limit that may be reached under unfavorable conditions, when the degree of saturation approaches 100 percent. It is an economic question if the design of flexible pavements should be based on the most unfavorable conditions or on some other more favorable condition; e.g., 90 percent saturation of the subgrade soil.

Mr. Benkelman also brought out that the bearing capacity of flexible pavement as affected by frost action has been evaluated by plate load tests. The bearing capacity determined in this manner when the load is applied relatively slowly and when very small excess pore pressures or none develop in the soil may be different from that governing the bearing capacity with respect to moving traffic loads when the loads are applied relatively rapidly and when high excess pore pressures may develop in the subgrade soil. The method proposed by the author takes these excess pore pressures in account and it has been shown that these excess pore pressures have a very large effect on the ultimate bearing capacity. It is possible that the results from the load tests cannot be used to predict under all conditions the bearing capacity of flexible pavement subject to rapidly applied loads such as moving traffic loads.