Flexible Pavement Design in Idaho

L. F. ERICKSON, Materials Engineer, Idaho Department of Highways

This paper reviews flexible pavement design methods used by the Idaho Department of Highways since soils tests were begun in 1938. California Bearing Ratio tests were used prior to World War II, but in 1942 an empirical soil formula similar to group index was adopted and used until 1950 for evaluating subgrade soils for flexible pavement design.

The Hveem stabilometer has been used for bituminous work since 1938, but construction of a kneading compactor in 1950 materially expedited the work. The paper by Hveem and Carmany (1) on factors underlying the rational design of pavements in 1948 led to adoption of resistance value criteria for the design of flexible pavements.

The paper reviews other criteria considered by Idaho in the design of flexible pavements. It concludes that after seven years use of resistance value criteria, this method is satisfactory and proper utilization of these data will do much toward reducing the number of subgrade, base, and surfacing failures on Idaho roads.

THE FIRST ROUTINE use of soil testing by the Idaho Department of Highways was started in 1938 with emphasis on compaction control. Previous use of soil test data had been limited to only a few isolated locations where it was evident that unusual conditions existed and further knowledge was desired. A soil survey and soil profile was made for each project following essentially the procedures set forth in AASHO T 86, "Surveying and Sampling Soils for Highway Purposes," and the soil was classified in accordance with the Bureau of Public Roads classifications. All test information was reported to design and construction engineers during this period and they became fairly familiar with the soil test data and noted some relationships between soil types and field performance.

During 1940 the laboratory began to evaluate the soil data and to recommend a total thickness of base and surfacing for use in design. At this time the evaluation was based almost entirely on the judgment of the evaluator and was seldom supported by information from field performance. Nevertheless, these evaluations did much toward making designers and construction personnel conscious of soil types and characteristics.

In general, the thicknesses proposed by the laboratory were accepted because the recommendations were for thicker base and subbase courses than had been provided previously. Some engineers, being familiar with particular local soils and conditions, were better able to correlate performance with soil types locally and in some instances increased the thickness above that recommended by the laboratory. A soil survey using essentially the methods outlined in AASHO T 86 is still in use today, and electrical resistivity equipment has been used to good advantage since 1956. In 1957, seismic equipment was purchased as a further aid in this work. All essential test data are shown on the soil profile, together with recommendations for total thickness of the pavement structure and any needed special treatment of subgrade.

Prior to 1942 the California Bearing Ratio test was made on nearly all soil samples, compacting the sample under a static load of 2,000 psi. Although it was realized that this loading did not give a density comparable to that obtained in the test for moisture-density relationship, it was believed that, after the 4-day soaking period, practically the same results would be obtained for CBR as those to be expected had comparable densities been obtained. About 1940, tests such as the centrifuge moisture equivalent, shrinkage limit, and hydrometer analysis were discontinued as routine tests. A linear shrinkage test was adopted in lieu of the shrinkage limit, as it was a more expedient test and measures equivalent properties.

EMPIRICAL SOIL NUMBER

North Dakota, in 1942, published a formula in which a soil number was computed utilizing the soil test data used to determine the BPR soil classification. The soil number thus obtained was reportedly correlated with service behavior. But there were many different opinions regarding the thickness of pavement structure to be provided. Also, to add confusion, there were inconsistencies in the evaluations from each individual evaluator and these inconsistencies proved to be embarrassing to the laboratory.

The empirical soil number offered a solution to this problem, as well as a means toward obtaining better correlation with field performance. The North Dakota formula was revised, using only those tests that were routine in Idaho. The Idaho soil number was correlated with previous evaluations, which were confirmed by road performance in local areas. The formula used in computing the soil number in Idaho has not been changed since its inception and is as follows:

$$\text{Ballast Number} = A + B + C + (D + E + F + G) + H \qquad (1)$$

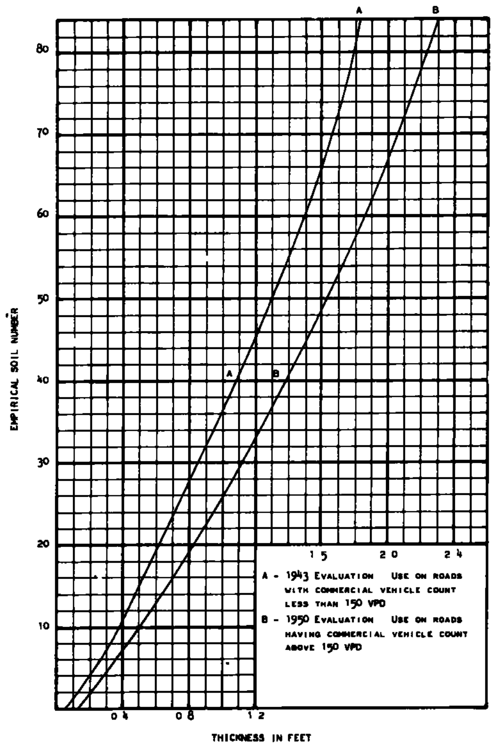


Figure 1. Design pavement thickness chart based on empirical soil number.

in which

$$A = \frac{\%\text{ pass No. } 10 - 50}{10} \quad (\text{if less than 50 percent pass, } A = 0);$$

$$B = \frac{\%\text{ pass No. } 40 - 30}{10} \quad (\text{if less than 30 percent pass, } B = 0);$$

$$C = \frac{\%\text{ pass No. } 40}{40} \quad (\text{if more than 40 percent pass, } C = 1);$$

$$D = \frac{\text{Liquid limit} - 15}{3} \quad (\text{if liquid limit less than 15, } D = 0);$$

$$E = \text{Plasticity index} - 5 \quad (\text{if plasticity index less than 5, } E = 0);$$

$$F = \frac{\text{Rose moisture equiv.} - 15}{10};$$

$$G = \text{Lineal shrinkage (use nearest whole number); and}$$

$$H = \frac{130 - \text{weight/ cu ft}}{3} \quad (\text{use maximum dry weight from moisture-density test}).$$

The factor C is used to reduce the numerical value resulting from adverse soils constants in granular materials in proportion to the amount of material passing the No. 40 sieve. This is because the more granular material there is in a sample the more stable the material should be regardless of adverse soil characteristics when considered as a subgrade material.

The application of this formula and the associated design curve was changed in 1950 to give about 25 percent increased total thickness of pavement structure. The greater thickness provided by this revision was based on results of an investigation of the behavior of bases and pavements placed just prior to and immediately after World War II. Both the original (1943) design curve and the 1950 curve are shown in Figure 1. It was recognized that the amount and type of traffic had a significant part in pavement performance, so a total commercial vehicle traffic count of 150 ADT was selected as the dividing point for design purposes.

There is a similarity between the Idaho empirical soil number and the group index, although they give only a general correlation. A comparison made several years ago indicated that a better correlation resulted when the weight per cubic foot and linear shrinkage values were omitted from the Idaho formula. These values are believed, however, to indicate properties that are significant in performance and their use has been continued in the computation of the Idaho soil number.

ADOPTION OF RESISTANCE VALUE CRITERIA

The California Bearing Ratio test was discontinued after World War II, as it was then realized that static compaction was unsatisfactory. Impact compaction, although it improved the test, required more time than permissible due to personnel limitations, if several specimens were prepared. Design charts proposing pavement thicknesses from the CBR test data appeared to give excessive thicknesses in the opinion of some of the engineers and this also discouraged its use.

Idaho had been using the Hveem stabilometer for testing bituminous surfacing for several years and in 1950 constructed a pneumatically-driven kneading compactor to be used in preparing specimens of bituminous mixtures for testing. It was soon learned that the compactor gave entirely different results for stability than did static compaction. This information was checked by testing unstable mixtures taken from existing pavements and it was determined that kneading compaction often gave stability values considerably below those obtained for static compaction on the same mixtures. Since 1950 there have been numerous occasions for testing stable and unstable bituminous mixtures from existing pavements and the kneading compactor has permitted the stabilometer to indicate the difference readily, whereas static compaction has rarely given the same indication. The criteria in use by the California Highway Department

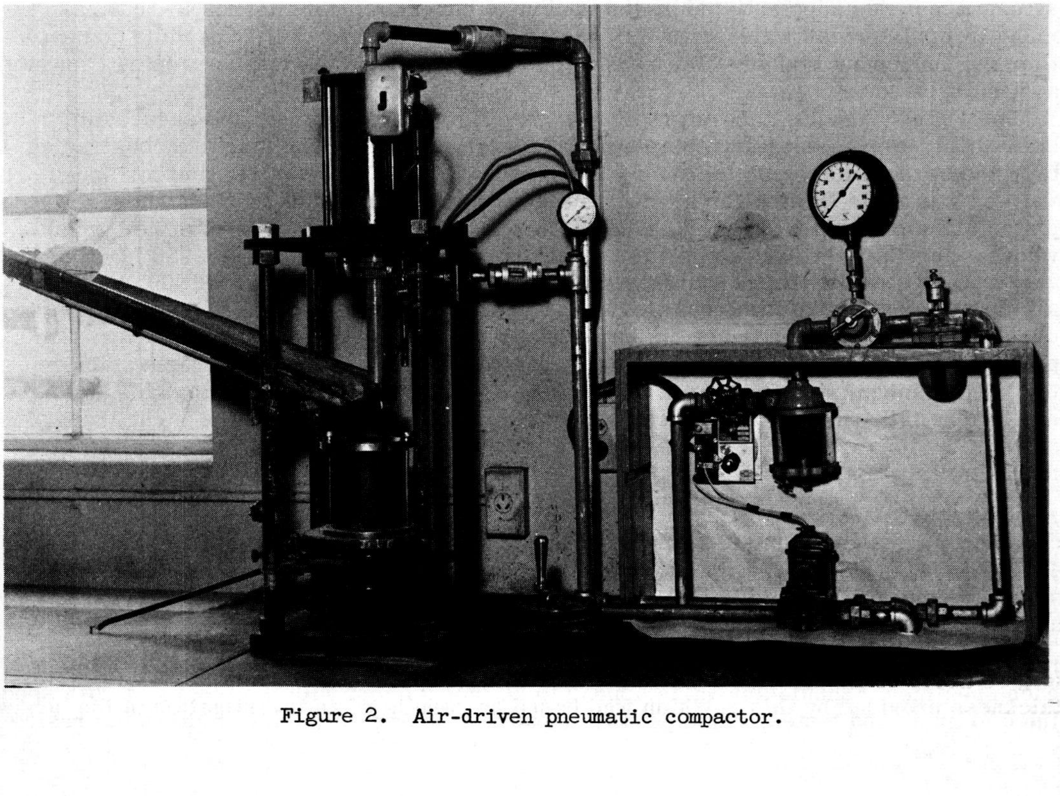


Figure 2. Air-driven pneumatic compactor.

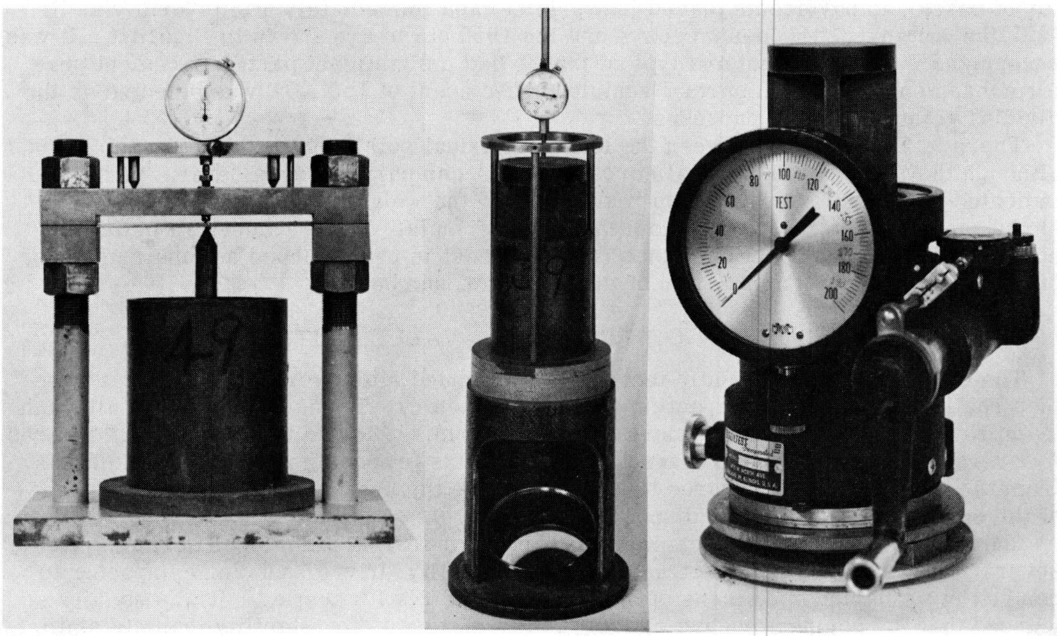


Figure 3. Expansion pressure frame (left), Washington visual saturation indicator (center), and Hveem stabilometer (right).

for bituminous mixtures were verified in every case investigated. The design procedure outlined by Hveem and Carmany developed considerable interest, and because of the information obtained from investigating unstable bituminous surfacings it was decided that the stabilometer method should be tried for soil testing. The need for a strength test was substantiated by the review of pavement performance in 1950, as many inconsistencies were noted. It was concluded, however, that kneading compaction and the Hveem stabilometer give reliable results when the test is properly conducted and interpreted.

The Forest Service Laboratory at Arcadia, Calif. had designed and successfully built an air-driven kneading compactor that was practical. The machine constructed by Idaho, except for electric timer and controls, air pressure valve, and air speed valves, was entirely manufactured in the Idaho highway shops using the double-acting piston principle of the Forest Service machine. Air was used to operate the turntable, whereas the Forest Service operated its turntable electrically. Since 1950 Idaho has constructed three of these machines. The latest (1957) cost less than $800, including all controls, with shop time charged at the rate of $4.50 per hour. Figure 2 shows one of these compactors set up in the soil laboratory. Figure 3 shows the expansion pressure frame, the Washington visual saturation indicator, and the Hveem stabilometer. Figure 4 is a trace of the foot pressure developed while compacting soil in a normal manner; that is, at 30 strokes per minute at 250-psi foot pressure. The trace, obtained by means of SR-4 straingage equipment, indicates that no impact is imparted to the soil specimen.

Preparation of test specimens is expedited by this method of test. Two men are able to prepare four soil specimens at varying moisture contents for each of the 25 or more samples tested each week. One additional man working half-time can increase this output to nearly 40 samples a week. These men weigh out the material for each specimen, moisten the sample and permit it to condition overnight, compact the specimens, determine saturation pressure, measure and weight specimens for density, measure expansion pressure, and compute the resistance value and moisture content of each specimen. Should any value appear to be inconsistent with the others, a fifth specimen is made and tested. This permits construction of a satisfactory curve for resistance value versus expansion pressure and resistance value versus saturation pressure. Bituminous mixtures are tested in another section of the laboratory; if the need should ever exist, a comparable output could be attained in this section.

DETERMINATION OF RESISTANCE VALUE, EXPANSION PRESSURE, AND SATURATION PRESSURE

The procedure used for determining the resistance value, saturation pressure, and expansion pressure follows the procedure developed by California, except that a compaction pressure of 250 psi is used. This is the unit pressure specified for sheepsfoot rollers in the 1950 Idaho specifications. This value was chosen because it was believed that greater pressures would result in too high densities and thereby possibly produce higher resistance values than should be depended on in relationship to the densities specified or attained in construction.

Each specimen is made using material from which material coarser than ¾ in. is removed and without replacing this oversize with an equal amount of No. 4 to ¾-in. material as is advocated in some other tests. Sufficient material is prepared and moistened to make at least five test specimens of the proper height and weight after compaction. This material is permitted to condition over night so that all of it has been permitted to absorb moisture. The moisture content of the material is then determined and sufficient material is weighed out for each specimen.

The first specimen is compacted at the existing moisture content, with the compactor applying a foot pressure of 250 psi for 140 strokes. The specimen is then placed on the Washington visual saturation indicator and compressed by static loading until five of the six segments appear wet. The load applied is recorded as the exudation or saturation pressure. The weight, height, and weight per cubic foot of the specimen are then determined. The specimen is then set aside and permitted to rebound.

A second sample is then moistened, adding approximately one percent water as indicated by experience, and the foregoing procedure is repeated. All specimens are permitted to rebound for approximately one hour. They are then placed in the expansion pressure frame and the stand adjusted to apply a surcharge load of about 0.1 psi to seat the stand. The mold is filled with water to about 1½ in. above the surface of the soil and the pressure developed by the soil in endeavoring to expand is determined after 16 to 18 hr by the deflection measurement of the calibrated bar. The specimen is then pressed from the mold into the Hveem stabilometer and the resistance value is determined by applying a vertical load at a strain rate of 0.05 in. per minute and reading lateral pressures for vertical loads of 500, 1,000, 1,500 and 2,000 lb. The vertical load is then reduced to 1,000 lb and the lateral pressure is reduced to 5 psi by means

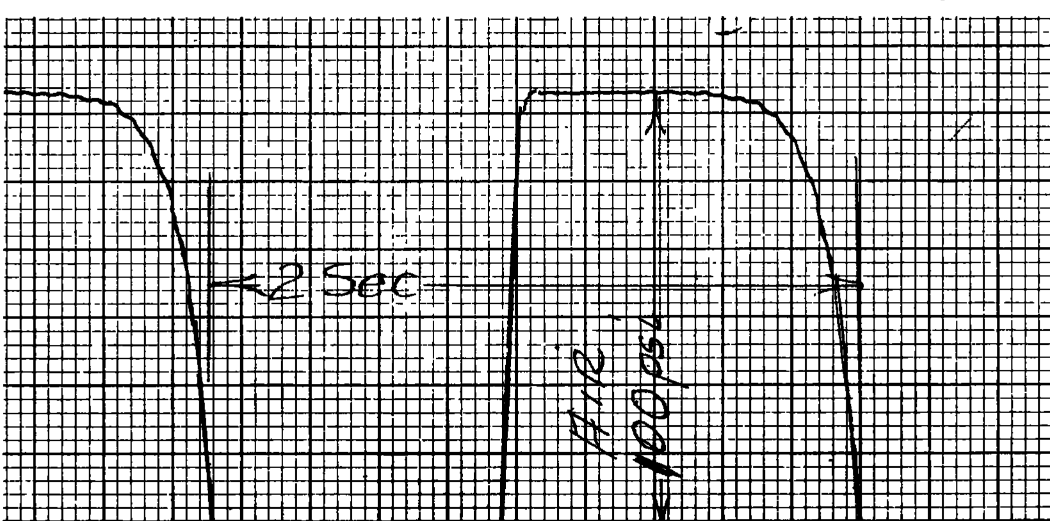


Figure 4. Foot pressure diagram for compactor.

of the displacement pump, permitting the vertical load to continue to drop. The displacement pump turns necessary to increase the lateral pressure from 5 psi to 100 psi are noted and recorded as the displacement. A moisture determination is made of the material so that the amount absorbed during the expansion pressure test and the moisture content at the time of the R-value test will be known. The resistance value (R value) is calculated from:

$$R = 100 - \frac{100}{\frac{2.5}{D}(P_v P_h - 1) + 1} \tag{2}$$

in which

D = Displacement turns;

P_v = Vertical pressure (160 psi at 2,000 lb); and

P_h = Horizontal or lateral gage pressure for 2,000-lb load.

Resistance values for each specimen are plotted against the corresponding saturation pressures and a smooth curve is drawn through the points. The R value for a saturation pressure of 5,000 lb is taken as the R value of the material, although a greater or lesser saturation pressure could be used. This is the value used by the California Department of Highways. Similarly, the expansion pressure is plotted against R values and the expansion pressure noted where R value and expansion pressure give equal total pavement thickness for an assumed traffic index of 7 and a 130-pcf unit weight of material. These curves are reproduced on the laboratory report (Fig. 5) submitted to designers and field personnel.

Select borrow, select base, and base materials are tested for R value and expansion pressure as previously outlined whenever the material is plastic or has properties that may be considered doubtful for quality, such as material subject to degradation. Such material will degrade in the compactor; therefore, 1,000 strokes at 250 psi have been applied to deliberately degrade the material before testing it for R value. Although this does not represent the end product of degradation in the field, it may nevertheless indicate how seriously the R value is reduced by degradation.

EVALUATING TRAFFIC

A traffic index is computed for each project, using data obtained from reports of the planning survey. The estimated traffic data projected for 20 years hence are presented in a design brochure prepared by the design engineer. This brochure includes essential information for the facility to be provided, including geometric sections of the highway and locations of frontage roads, interchanges, and separations. Until 1957 the traffic index was computed by the formula presented by Hveem and Carmany (1). A more recent formula has been developed (it is understood that is is to be included in a proposed manual of the AASHO Committee on Design), as follows:

$$TI = 1.35(EWL)^{0.11} \tag{3}$$

in which EWL = 5,000-lb equivalent wheel loads.

Idaho traffic data have been reviewed twice since 1950 and it is noted that the average axle loadings have increased from slightly less than 12,000 lb in 1950 to about 14,500 lb in 1956. This increase results in a large increase in the EWL value for the same number of vehicles. Using available traffic data, the traffic index has been computed for varying numbers of heavy vehicles; that is, vehicles having axle loadings of more than 10,000 lb. Figure 6 gives the traffic index to be used for these various groupings of vehicles and a design chart plotting total thickness against R value for varying traffic index numbers assuming a cohesiometer value of 100 for granular material. The total depth of pavement structure to be provided is computed from:

$$T = \frac{0.095\ TI\ (90 - R)}{\sqrt[5]{C}} \tag{4}$$

in which

T = Total thickness, in inches;
TI = Traffic index;
R = R value; and
C = Cohesiometer value assigned various surfacing materials, i.e., P.C. concrete, plantmix, roadmix, cement treated base, etc.

The traffic index is varied from 3 to 11.5 This provides for roads carrying only a couple of heavy vehicles per week to a highway that would carry 2,900 or more per day.

Copies to: STATE OF IDAHO IH-803-11-57

Highway Engr.
District Engr.
Resident Engr.
B.P.R.
File

DEPARTMENT OF HIGHWAYS

Materials Laboratory

BOISE

Lab. No. 125855

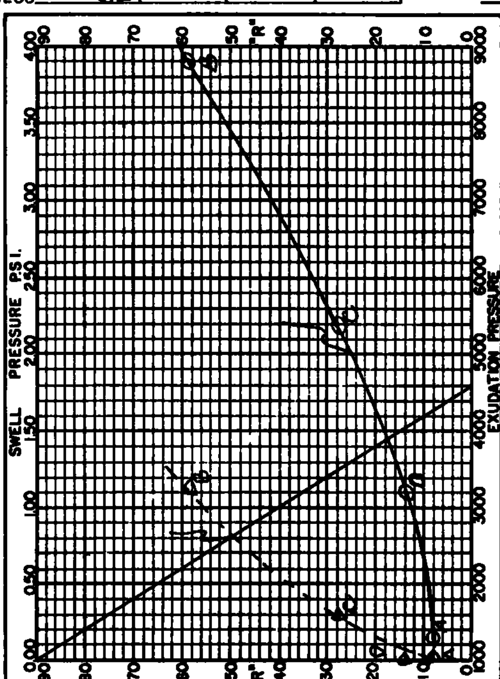
Report of Tests on SOIL Embankment & Subgrade

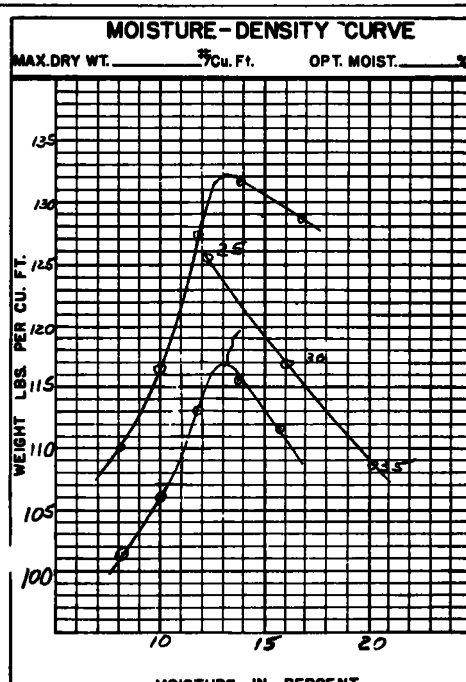
Project I-IG-1032(5) Blackfoot-Bonneville Co.Line County Bingham
Submitted by J. Alford Ident. No. JA/4549-2213/8-P
Layer No. 4 Source No. ____ Depth, feet 0.6'-3.0' Station 1135, Rt. Lane
Description of Soil Silt and Gravel Date Sampled 11-27-57 Received 12-2-57

Mechanical Analysis % Passing				
3" Sq.				
2" Sq.	100			
1" Sq.	92			
3/4" Sq.	87			
1/2" Sq.	81			
No. 4	76			
No. 10	76			
No. 20	74			
No. 30	74			
No. 40	72			
No. 50	71			
No. 100	61			
No. 200	52			

Soil Constants

Liquid Limit 22 "R" Value 25
Plastic Limit 18 Swell Pressure Psi 0.82
Plasticity Index 4 Eq. "A" No. 18
Field Moist. Equiv. 20.6 Texture Class'n Silt loam
Linear Shrinkage 4.0 Soil Class'n A-4(3)
Specific Gravity 2.68
Remarks





This report covers only material as represented by this sample and does not necessarily cover all soil from this layer or source.

Date Mailed December 10, 1957

L. F. Erickson
Materials Engineer

Figure 5. Soil report form.

TRAFFIC INDEX	HEAVY VEHICLE A.D.T.	
3.0	2 or less/week	Residential streets.
4.0	1 or less/day	Frontage roads with no trucking.
5.0	1-4	Frontage roads not serving warehouse or industrial traffic.
5.5	4-12	County roads of very light traffic volume.
6.0	12-20	Normal highway traffic including primary county roads with normal type and distribution of heavy vehicles.
6.5	20-50	
7.0	50-80	
7.5	80-140	Frontage roads serving warehouse and industrial traffic.
8.0	140-275	
8.5	275-550	
9.0	550-850	
9.5	850-1350	
10.0	1350-1750	
10.5	1750-2350	Very heavy commercial traffic. Industrial traffic such as ore and log trucks of abnormal numbers with supporting data furnished.
11.0	2350-2900	
11.5	2900 & over	

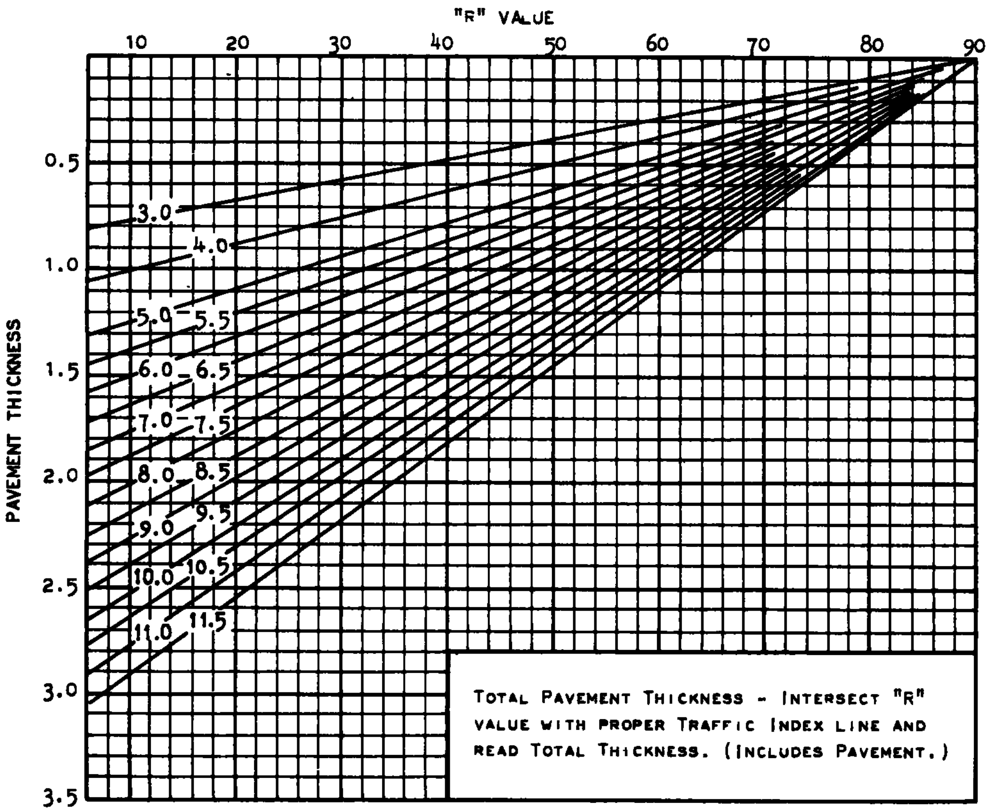


Figure 6. Design pavement thickness chart—traffic index and resistance value.

The normal variance of the traffic index used in Idaho ranges from 6.0 to 10.0. Values less than 6 are used for frontage roads not serving heavy vehicles and for some county roads of extremely light traffic volumes. A traffic index of 3 is used where only passenger car and pickup traffic use the roadway or street. The state has a few roads where most of the traffic involves heavy vehicles, such as logging trucks or ore trucks, which are all loaded to maximum legal axle capacity. In one case, approximately 600 ore trucks daily, each 5-axle and carrying a gross load of 72,000 lb travel one side of a secondary highway for a distance of about 12 miles from late spring when load limit restrictions are lifted until the mine is shut down in the winter. Highways carrying this type of traffic must be given special consideration, bearing in mind the period of the year when the highway is being used and providing a design adequate for that period. Idaho has many miles of county highways that serve only limited traffic; some have a traffic count of less than 50 vehicles per day. Many of these roads are located in remote areas and although a need does exist for a serviceable road, available funds permit only a minimum facility. Due to the extremely low traffic volume, many of these roads have been designed using a traffic index of 5 or 5.5 and utilizing selected local material to avoid crushing operations or at least to keep such processing to a minimum.

Reductions in total thickness of pavement structure due to the use of cohesiometer values for cement-treated base or plantmix have rarely been utilized. This concept is new to Idaho and although this method of design probably will be adopted, traffic increases in recent years have so far exceeded expectations that there is an inclination to forego this reduction in thickness as an additional safety factor.

OTHER FACTORS CONSIDERED

Several other factors are considered in the design of flexible pavements. Fine-grained plastic soils used in the subgrade are covered with a layer of dense-graded aggregate or sand to prevent intrusion of the soil into the interstices of coarser base material. Normally this blanket course is about 3 in. thick. For fifteen years, Idaho has used as a guide criteria that any soil possession more than 70 percent passing the No. 200 sieve and having a P.I. greater than five and a linear shrinkage greater than five, should be blanketed with a dense-graded aggregate or sand. These criteria may be too conservative, but to date it has not been possible to determine better standards. Generally, where sand and gravel are available a blanket material is not required, because the soil types in areas containing sand and gravel deposits are generally sandy in nature and rarely very plastic. Perhaps of greater importance, is the fact that the sand-gravel deposits generally contain sufficient materials passing the No. 10 and No. 40 sieves to serve adequately as a blanket material. These blanket course materials are required to have 25 percent passing the No. 10 sieve if ¾-in. maximum and 20 percent passing the No. 10 sieve if 2-in. maximum. This requirement makes the production of stone aggregates costly when this is the only material available for use.

The Idaho standard specifications require that unsuitable material at grade points be excavated and replaced with approved material, preferably granular in nature. But there are a variety of opinions as to what constitutes "unsuitable" material. Therefore, included on the soil profile are notes and limitations of the areas to be excavated, giving depth of excavation below finished grade. These areas are also shown on the construction plans and whenever practicable the plans provide that the excavated area be sloped for natural drainage. Close attention to this detail is worthwhile and avoids subgrade trouble in critical areas where failures have so often occurred. Figure 7 is an artist's sketch illustrating the method of excavating and backfilling.

Where frost is not a problem, a select soil may be used to backfill grade points deemed necessary to be excavated. Where frost troubles are common, a granular material is required even though base must be produced for this purpose. Special attention is given to all drainage. Depths below subgrade which are commonly excavated and backfilled vary from 0.5 ft where frost is no problem to more than 1 ft where frost boils are common. This practice is not perfect in its application as frost boils do occur at other locations than grade points.

The practice of excavating grade points is controversial and has been subject to many

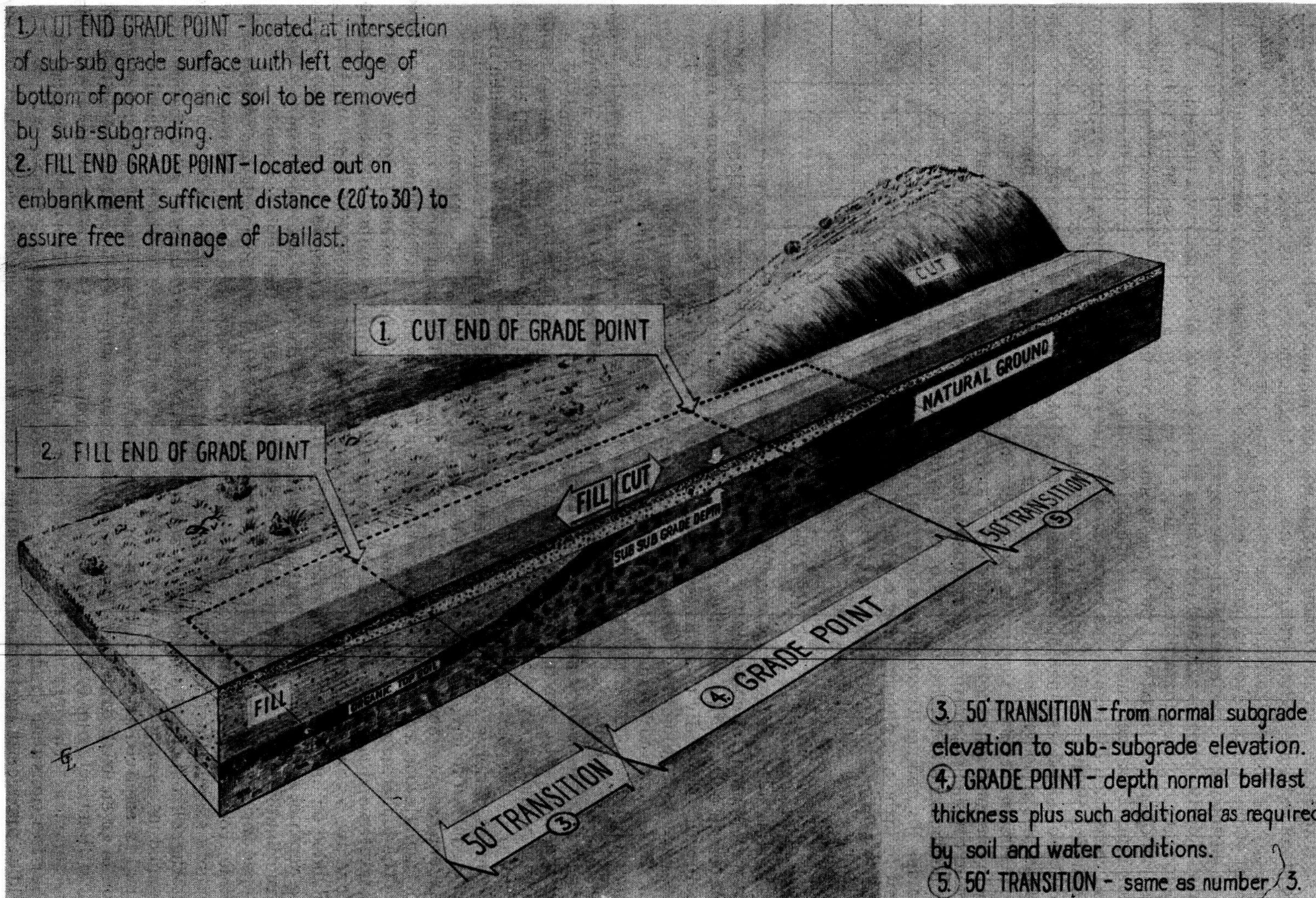


Figure 7. Method used to reinforce grade points.

Date

Project No.: I-IF-1032(5)
Section Blackfoot-Bonneville Co. Line
County Bingham

STATE OF IDAHO
DEPARTMENT OF HIGHWAYS
Materials Laboratory
BOISE

Sheet 1 of 1
Projected Traffic Data
Total V/Day 10640 1975
Commercial (Does not include panels & pickups) 1120

Curve "B"   Traffic Index 9.5

Station	Lab.No.	Layer No.	Depth	Class'n	L.L.	P.I.	FME	L.S.	% Passing			Moist-Dens.		Eq. "A"	"R" Value	Expansion Pressure psi	Recomm. Thick.	Blanket
									#10	#40	#200	W/C.Ft	Opt. Mois.					
1135 Rt.	125855	4	0.6'-3.0'	A-4 (3)	22	4	21	4.0	76	72	52	116.8	13.0	18	25	0.82	2.0	No

GENERAL REMARKS: This evaluation is based upon the assumption that the soils evaluated will be used in the top subgrade layers and that standard drainage and compaction will be provided. The above recommendations for total thickness are considered for average climatic conditions, traffic, availability of base and surfacing materials, etc. One amply designed section over a group of soils is considered good practice for the sake of uniformity of section thickness.

Figure 8. Soil evaluation form.

extremes. Designers in a district having very little difficulty with frost boils may provide better than 1 ft of subgrade excavation, whereas other districts having a definite problem may ignore or only partially accept the criteria because some engineers believe the boils are impractical of correction. Progress is being made, however, and with these areas shown on the plans a more effective treatment probably will be obtained. The subgrade treatments designed recently are much more reasonable than when first proposed.

STANDARD DESIGNS

Idaho has endeavored to standardize the thicknesses of bituminous surfaces and base courses as much as possible. The interstate highways are designed to have a future 0.1-ft course of plantmix with 0.3 ft in two courses provided in the original construction. Primary highways will have 0.3 ft in two courses for roadway widths of 40 and 44 ft, and 0.2 ft in one course for roadways 34 ft wide. Secondary roads will be given 0.2 ft of bituminous surfacing of either plantmix or roadmix, depending on the traffic count. All bituminous pavements are constructed full width from shoulder to shoulder.

Base courses immediately below the bituminous surfacing will consist of 0.4 ft of ¾-in. maximum base in two courses for the wider roads, and 0.3 ft in a single course for the 24- and 28-ft roadbeds. Other subbase and base courses needed will be governed by the total thickness to be provided and whenever possible will be of pit run material but having characteristics very similar to the high-type base material for plastic index and sand equivalent. An endeavor is made to always provide a free-draining material in the base courses, including any select materials used. The base courses are also constructed full width. If the select material or base possesses any plasticity, R-value determinations are made to determine the suitability for use. The adoption of the sand equivalent test in the Idaho specifications in 1956 has aided materially in providing apparently free-draining material.

An example of the soil evaluation presently furnished the Design Engineer is shown in Figure 8. This evaluation sheet includes the group index number as a part of the soil classification by noting it in parentheses. The empirical soil number is reported, as well as R value and expansion pressure. A total thickness of pavement structure is recommended for the traffic index shown at the head of the sheet and is generally based on R value or expansion pressure. However, if the soil number gives a greater total thickness for an area subject to frost damage, the greater thickness is used. The design engineers and district materials engineers are getting together and establishing the thickness to be provided for frontage roads, ramps to interchanges, and separation structures. This information is included in the project soil report, which covers the soil survey, soil profile, sub-subgrade treatments, and total flexible pavement design thickness for the project.

EVALUATION OF DESIGN PROCEDURES

Idaho has been using R value as one basis of design since 1950 and has been using the Hveem stabilometer for determining the stability of bituminous mixtures since 1938. Without the kneading compactor, however, the results obtained in stability tests and R-value tests in all probability are misleading. The combination of the compactor and the stabilometer makes a most useful and rapid instrument for testing all material used in the construction of a pavement structure from the subgrade up to and including the bituminous surfacing. Much greater confidence is now placed in the R-value method of design rather than on the empirical soil number. The resistance value test is considered no more subject to experimental error than CBR. It is sufficiently rapid to permit at least four values to be determined for each soil sample, permitting the selection of an R value from a curve rather than from a single determination. It is felt that this test and the method of application will do much toward reducing the number of failures of subgrade, base, and surfacing on Idaho roads.

An attempt is now being made to measure the permeability of granular soils when testing for expansion pressure. The rate of flow through the sample is a variable and by means of measuring time and drop in the water level within the mold, a value for permeability can be computed. This value may not be as accurate as desired, but does appear to indicate the permeability within a reasonable range. It has been found that some A-3 soils have very low permeability, whereas permeabilities in apparently dense-graded material have been very rapid. Not many data are available as yet, but this test is being conducted on any soil considered possibly permeable, as well as on all select borrow and base materials tested for R value.

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   (c) Test Method T-8-54, "Standard Method of Testing Soil for Resistance Value."
   (d) Test Method T-9-54, "Standard Method of Testing Bituminous Mixtures for Relative Stability."
   (e) Test Method T-15-54, "Standard Method of Test for Degradation of Aggregates."

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