Application of Waterways Experiment Station 7257-kg Vibrator to Airport Pavement Engineering

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For many years the airfield pavement industry has been searching for a suitable nondestructive method that would eliminate the necessity for borings and test pits. This paper describes the use of the Waterways Experiment Station 7257-kg (16-kip) vibrator for the evaluation of load-carrying capacity and the design of bituminous concrete overlays for highly variable flexible pavements at commercial airports. The primary purpose of using dynamic testing was to provide a rapid, nondestructive, and independent system of measurement of existing pavement strength. The vibrator is electrohydraulic and can apply loads up to 66.7 kN (15 000 lbf) on a 45.7-cm-diameter (18-in-diameter) plate at frequencies of 5 to 100 Hz. Primary measurements included dynamic stiffness testing, borings with California bearing ratio tests, and condition surveys. Dynamic stiffness was correlated with physical condition and types and thicknesses of pavement and subgrade to determine allowable gross loads and overlay thicknesses. The study shows that the stiffness concept in which a large vibratory load is used relates well to conventional (California bearing ratio) methods provided that sufficient conventional data are available at a limited number of locations representing the range of conditions. The immediate potential values of this method are speed of field operation, unexpected ranges of strength, and convenience of a single parameter expression of overall pavement and subgrade strength. Potential improvements in dynamic nondestructive methodology include use of deflection basin data and relation of stiffness to a theoretical basis.

Dynamic nondestructive testing of pavements has been undergoing considerable development since the 1950s. This paper describes the use in an actual design project of the third generation U.S. Army Engineer Waterways Experiment Station (WES) equipment as developed under U.S. Air Force and Army research (1) and used in the Federal Aviation Administration (FAA) research in the early 1970s (2). Most of the data and methodology given here are from investigations by SITE Engineers, Inc., at Philadelphia International Airport in 1973. Reference is also made to information obtained from WES studies for the FAA and to investigations by SITE Engineers, Inc., at Albany County, New York, Airport and Oakland-Pontiac, Michigan, Airport. Most of the pavements involved were flexible or semiflexible.

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The purpose of the investigations was to evaluate the existing strength and to design bituminous concrete (BC) overlays for expected increases in aircraft size and frequency.

Specific objectives of the dynamic testing were to

1. Determine a single number parameter at individual locations that would express the relative strength of the highly variable existing systems involving pavement, base, and subgrade in depth;
2. Determine the existing strength and overlay requirements by a semiindependent system of measurements;
3. Obtain better (more closely spaced) coverage by actual tests than was practicable with conventional destructive tests; and
4. Expedite the investigations by minimizing shutdown times to aircraft operations and reduce the cost of the study.

EXISTING CONDITIONS

Philadelphia International Airport is built in the floodplain of the tidal Delaware River and has been under development since the 1920s. The original surficial soil was alluvial organic silt having California bearing ratios (CBRs) on the order of 1 to 5 percent and thicknesses ranging from 3.05 to 15.24 m (10 to 50 ft). Underlying this soil are very strong granular and cohesive soils. Overlying the organic silts are random fills of granular material; loose, hydraulically deposited sandy silts; and cinders.

The airport was built in stages above the random fills during a 40-year period. It was built to various criteria and with a number of different types of pavement sections and overlay thicknesses. Within the project area, the existing pavements were composed of materials as given in Table 1. Figure 1 shows a plan of the project area and the location of the types of pavement.

The physical condition of the flexible pavements varied from very good (no major defects) to very poor (continuous deep alligator cracking or significant rutting in the wheel-path areas or both). Previous traffic on

Figure 1. Project plan.

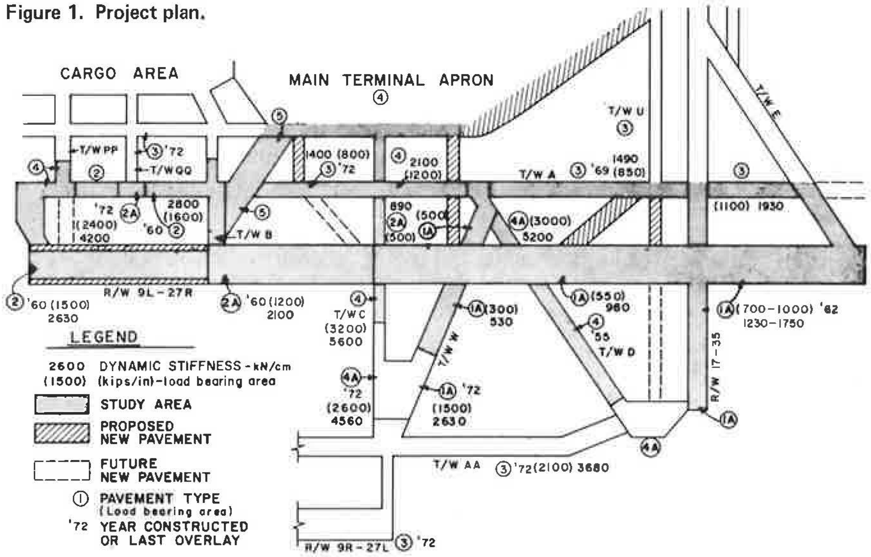
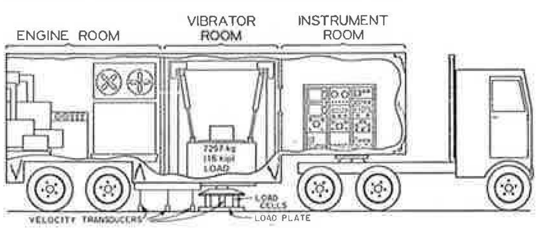


Table 1. Pavement composition.

Pavement	Surface	Stone Base	Select Fill	Random Fill	Overlay
1	5-cm BC	13 to 25-cm CM	0.7 to 0.9-m C	0.9 to 2.4-m S or SL + CL or both	
1A	5-cm BC	13 to 25-cm CM	0.7 to 0.9-m C	0.9 to 2.4-m S or SL + CL or both	5 to 33-cm BC
2	10-cm BC	25 to 30-cm M	1.5 to 3.0-m GS	0 to 1.5-m S or SL + CL or both	
2A	10-cm BC	25 to 30-cm M	1.5 to 3.0-m GS	0 to 1.5-m S or SL + CL or both	5 to 15-cm BC
3	28 to 33-cm BC	8 to 15-cm WGM	0.7 to 1.5-m G or G + S	0.7 to 1.2-m S or SL + CL or both	
4	30-cm PCC	0	0.9 to 1.5 G or G + S	0.7 to 0.9-m S or SL + CL or both	
4A	30-cm PCC	0	0.9 to 1.5 G or G + S	0.7 to 0.9-m S or SL + CL or both	5 to 20-cm BC

Notes: 1 cm = 0.394 in. 1 m = 3.28 ft.
BC = bituminous concrete; C = cinders; CL = coal; CM = coarse material; G = gravel; GS = gravelly sand; M = macadam; PCC = portland cement concrete; S = sand; SL = silt; WGM = well graded material.

Figure 2. Nondestructive testing equipment.



most of the pavements in the project area had been dual and dual tandem jet aircraft over 10 to 15 years.

PREVIOUS TESTS

During a 1970 preliminary evaluation, the pavements were tested with a model 400 road rater to assist in determining locations for destructive tests. In November 1972, WES made dynamic tests for their FAA research study on the newly constructed pavements of runway 9R-27L and taxiway AA. At the request of the Philadelphia Division of Aviation, additional tests were made at other locations, principally where destructive tests including CBRs had been made in 1970. In June 1973, WES returned to make research dynamic and destructive tests (through small-aperture borehole techniques) (3) at two additional locations on pavements completed in 1972. At this time, by contract with the city of Philadelphia and as directed by SITE Engineers, Inc., WES also made dynamic tests on the project reported here.

FIELD INVESTIGATIONS

Dynamic Testing Equipment

The WES 7257-kg (16-kip) vibrator, which is an experimental prototype model, is housed in an 11-m (36-ft) semitrailer that contains supporting power supplies and automatic data recording systems. The vibrator and mass assembly consists of an electrohydraulic actuator surrounded by a 7257-kg (16 000-lb) lead-filled steel box. The actuator uses up to a 5.1-cm (2-in) double-amplitude stroke to produce a vibratory load ranging from 0 to 66 723 N (0 to 15 000 lbf) with a frequency range of 5 to 100 Hz for each load setting.

Major items of electronic equipment are: a set of three load cells that measure the load applied to the pavement, velocity transducers located on the 45.7-cm-diameter (18-in-diameter) steel load plate and at points away from the load plate that are calibrated to measure deflections, a servomechanism that allows variation of frequency and load, an X-Y recorder that produces load versus deflection and frequency versus deflection curves, and a printer that provides data in digital form. Figure 2 shows the schematic view of the vehicle, major systems, and detection devices.

With this equipment, the vibratory load can be varied at constant frequencies and load versus deflection can be plotted. These load-deflection data are used to compute the dynamic stiffness modulus (DSM) for a pavement structure. The frequency can be varied from approximately 5 to 100 Hz at constant force levels to produce the frequency response of the pavement structure. Also, at any selected load or frequency, the deflection basin shape can be obtained.

Selection of the WES 7257-kg (16-kip) vibrator as a standard test to produce the DSM results has been somewhat arbitrary but is based on results of earlier research studies. The vibrator must be capable of applying static and dynamic loads sufficient to stress the entire pavement section under consideration. Also enough dynamic force must be applied to produce deflections large enough to be accurately measured. Studies over instrumented pavement sections have shown that the stress distribution with depth at a loading frequency of 15 Hz is nearest that of slowly moving wheel loads (with that at other frequencies between 5 and 50 Hz). The 45.7-cm-diameter (18-in-diameter) load plate with contact area of 1638 cm^2 (254 in^2) was selected because it approximates the single-tire contact area of most large jet aircraft.

Test Procedures

A dynamic stiffness test is performed by centering the test apparatus over the test location, lowering the contact plate, and slowly sweeping the dynamic force to a maximum of 15 kips (66.7 kN). The load-deflection data are plotted automatically in graphical form as the test progresses. For deflection basin shape measurements, the selected load is briefly held constant and the data are printed out in digital form and then manually.

Dynamic Investigations

The nondestructive tests consisted primarily of DSM measurements but did include a few variable frequency and many basin shape measurements.

The initial tests were made on closely spaced points along widely spaced lines transverse to the centerlines. The locations for these lines were selected from a study of the available data concerning the type and thickness of existing pavement, base, and subgrade-in-depth sections. From these tests at Albany County and Oakland-Pontiac airports, the locations for tests along longitudinal lines were selected. At Philadelphia International Airport, subsequent tests were made adjacent to previous and concurrent borings and test pits, in distressed areas, along certain longitudinal lines, and at other points of particular interest. A partial investigation plan for Philadelphia International Airport is shown in Figure 3.

Further use of stiffness testing was made at Philadelphia International Airport by performing tests along several transverse lines after several series of passes with a 32-Mg (35-ton) vibratory compactor and a 45-Mg (50-ton) pneumatic-tired proof roller. The purpose of these tests was to determine whether changes in the strength of the pavement and subgrade system occurred because of the two types of rollers.

More than 650 stiffness tests and 760 deflection basin tests were made at Philadelphia International Airport in 6 days.

Conventional Investigations

At Philadelphia International Airport, a comprehensive investigation into destructive testing had been authorized to provide conventional information for the design of overlays, new construction, reconstruction, special treatments at junctures of existing and proposed pavements, and other related items. Details of the procedures and results are given in the report (4) and include the following: (a) deep borings (to below the organic silt) at approximately 305-m (1000-ft) centers; (b) shallow borings about 4.6 m (15 ft) deep and core borings at about 61-m (200-ft) centers longitudinally and at about 7.6-m (25-ft) centers transversely at pavement intersections; (c) CBR and nuclear moisture-density tests in boreholes and test pits; and (d) a detailed visual condition survey including rut depth measurements at typical locations and complete defect mapping in selected areas. Typical locations of these types of investigations are also shown in Figure 3.

TEST RESULTS

Adjustment of Dynamic Data

From the load deflection plot, the WES method of determining DSM is to calculate the inverse of the slope of the straight-line portion of the curve (Figure 4). The resulting value is in kilonewtons per centimeter. A stiffness value may also be calculated for any point along the plot, and, as can be seen on the typical graph for a flexible pavement (Figure 4), the plot is usually concave upward. This yields higher values at points below the straight-line portion. The amount of curvature is believed to reflect the relatively higher rigidity of the surface and base materials rather than that of the overall pavement, base, and subgrade system. Also, the stronger the pavement is, the flatter the slope is. For the purpose of these investigations only DSM as determined from the straight-line portion was used.

Another correction that should be made to put all the data on a more common basis is the adjustment for temperature of the BC to a uniform temperature such as 21.11°C (70°F). Methods available for making this correction necessitate either direct measurement of the temperature within the pavement or estimates of an average pavement temperature by measurements of the surface temperature during testing and knowledge of the average air temperature for several preceding days. Tentative procedures recommended by WES are given elsewhere (2, 5).

DSM Test Results

The corrected stiffness values were presented in a manner to assist visual assessment of the variations, note where changes in patterns occurred, and aid in selecting typical values for further analysis. The first step was to evaluate the transverse sections taken at selected stations along the major pavements that represented the various pavement and subgrade conditions. As shown in Figure 5, there were major variations in stiffness both longitudinally and transversely. The runway had been built in three stages, and the center 45 m (150 ft) of the earlier stages had been overlaid at various times to improve the load-bearing capacity. From the data for stations 36+14 and 90+16, the thickness of BC would appear to be the major factor affecting stiffness. However, examination of the data for station 119+04 reveals that, even where the thickness of BC is uniform, the stiffness near the outer edges is only about 65 percent of that in the central area. Therefore, a second cause of variation in stiffness must be the effect of additional compaction of the base and subgrade by traffic. Weakening of nontraffic areas due to frost action is also possible. The destructive tests indicated no such magnitude of difference in base and subgrade strength. Another cause of variation appears to be a deterioration in overall strength in the vicinity of the wheel paths. This may be seen particularly at stations 90+14 and 119+04.

Examination of the taxiway sections revealed similar conditions especially for the western end, which is the only area that had received a large volume of jet traffic before testing. The areas consisting of 1-year-old 30.5-cm (12-in) BC pavement (taxiway W to taxiway B) revealed rather large differences that are probably due to a combination of nonuniformity in the strength of the old fills and the thickness of the granular fill from the recent construction.

Examination of the longitudinal profile of the runway (Figure 6) revealed further the effect of subgrade conditions to depths of at least 3.05 m (10 ft). On the runway, the western extensions included 1.52 to 3.05 m (5 to 10 ft) of controlled granular fill and had much higher DSMs than did the original portion, which has only 0.6 to 0.9 m (2 to 3 ft) of cinders over variable sand and silts. The eastern end contains some sand fill and tested somewhat higher than the central portion between runway 17-35 and taxiway C. The profile of taxiway A exhibited similar characteristics in that the western end is strongest and the thickness of reasonably dense granular fill appears to have the greatest affect. Two other observations were made.

1. Along taxiway W south of runway 9L-27R, the subgrade conditions are relatively uniform, but the thickness of BC in the wheel paths increases from about 13 to 76 cm (5 to 30 in) going southwest. The profile revealed an increase in stiffness of from about 700 to 3500 kN/cm (400 to 2000 kips/in).
2. On taxiway C, a difference of about 1050 kN/cm (600 kips/in) was noted from where there is only 30.5 cm (12 in) of portland cement concrete to where there is an overlay of about 13 cm (5 in) of BC.

For the Philadelphia International Airport project, longitudinal runs were not made; to analyze for representative stiffnesses in each area, a contour plan was drawn from the data. For Albany County and Oakland-Pontiac airports, profiles were used and the data were statistically analyzed to assist selection of significantly different areas. Low average DSM values for the load-bearing areas at Philadelphia International Airport are shown in Figure 1.

Deflection Basin Results

Typical deflection basins are shown in Figure 7 and indicate that the deflection slopes generally vary most significantly from the edge of the loading plate to sensor 2 15.24 cm (6 in) from the edge. Comparison of the basin shapes to pavement sections showed similarities among dissimilar pavement, base, and upper subgrade sections and differences among similar sections. Although they have not been analyzed in detail, the differences in shape are suspected to reflect the condition and strength of the lower portions of thick flexible pavements or the base and top of subgrade. Limited analyses show that there are general trends of increasing steepness of slope with decreasing DSM and that the trend varies with pavement composition. At Oakland-Pontiac Airport, a definite re-

Figure 3. Partial investigation plan.

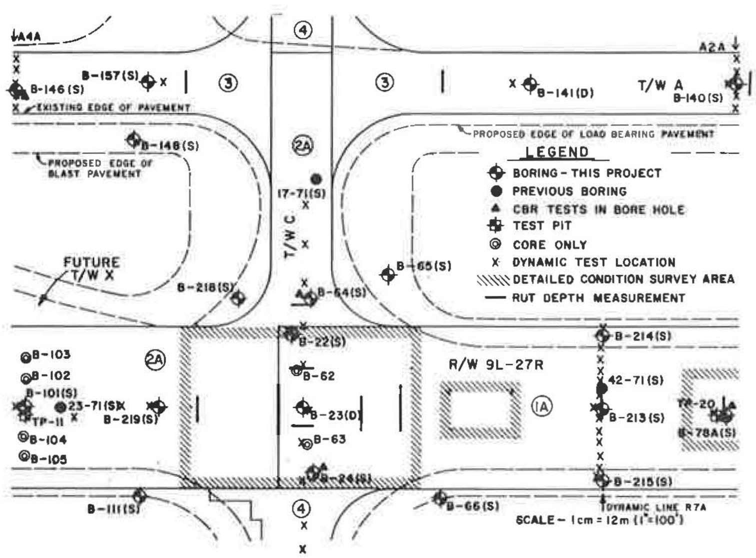
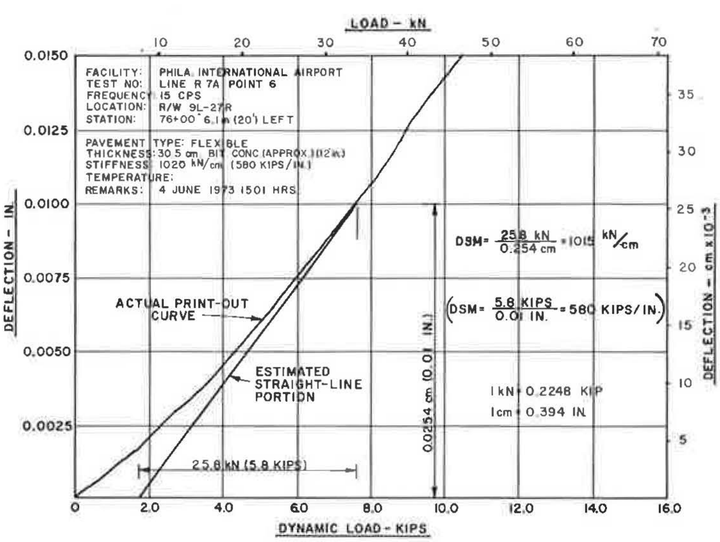


Figure 4. Field load deflection graph and DSM calculation.



lationship was found between the slopes and the condition of the soil-cement base.

ANALYSIS

Evaluation of Allowable Load-Carrying Capacity

At each location where adequate thickness and CBR data were available, an allowable gross airplane load was calculated. The evaluation was based on FAA criteria in effect in 1973 for 6000 to 12 000 equivalent critical departures (ECDs)/year (design life of 120 000 ECDs) of DC-8-63 aircraft. This equals approximately 40 000 actual departures/year for the 1975 air carrier jet traffic mix at Philadelphia International Airport. Because measured subsoil CBRs varied from less (1 percent) to more (1 to 50 percent) than assumed in the FAA procedures (3 to 20 percent), it was necessary to develop a thickness versus CBR curve similar to that used by the U.S. Army Corps of Engineers (6).

The computed allowable loads were plotted against the DSMS, and a relatively good correlation as shown in Figure 8 was obtained.

Selection of Design DSM Values

For Philadelphia International Airport, conservative existing DSM values were selected for each of the more than 50 analysis areas to be overlaid by examining the contour plans, profiles, and sections. The analysis areas were determined on the basis of known differences in pavement and subgrade conditions, estimated future traffic volume, and offsets from centerline in accordance with keel section design concepts. Traffic volumes in terms of ECDs were developed by analyzing the aircraft

Figure 5. Transverse DSM sections on runway 9L-27R.

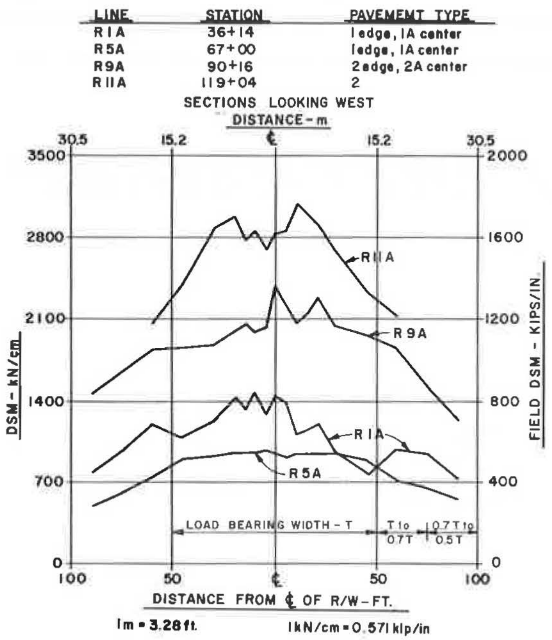


Figure 7. Typical deflection basins.

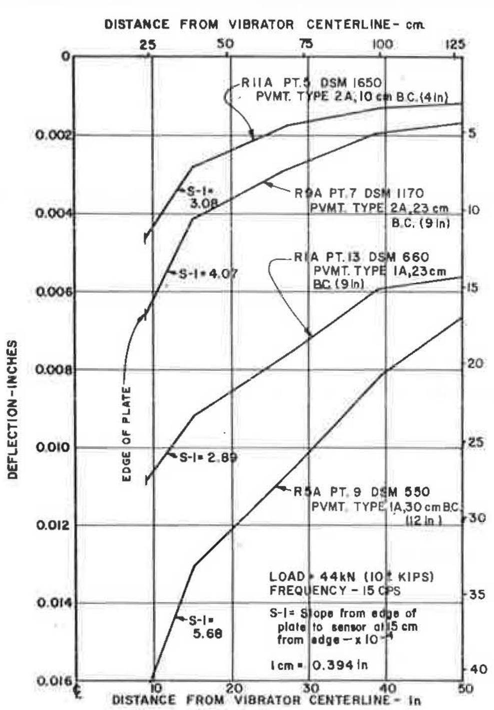


Figure 6. Profile of runway 9L-27R.

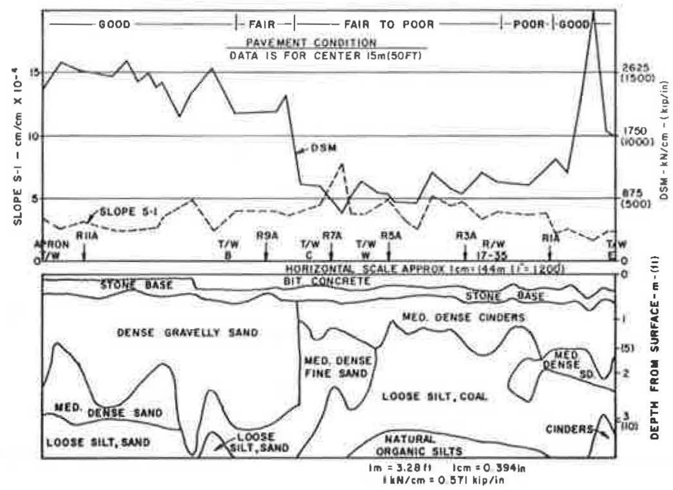
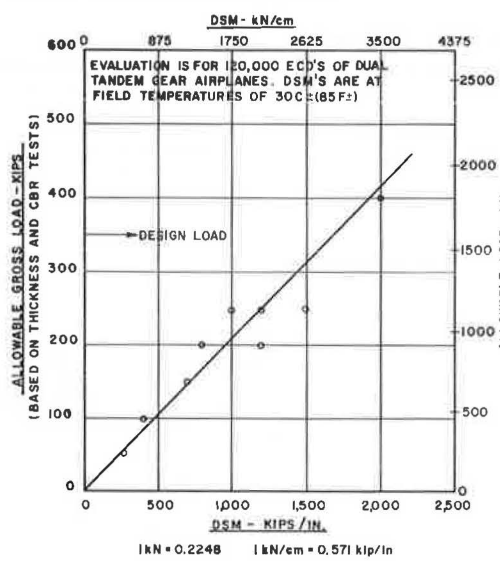


Figure 8. DSM versus allowable load.



mix in each area and coverage versus actual departure ratios (7).

To use the DSM for overlay computations, we had to select design criteria values for the several levels of traffic volume. The DSM for a 1557-kN (350-kip) design load from Figure 8, 2977 kN/cm (1700 kips/in), was one possible value for 6000 to 12 000 ECDs/year, but the DSMs from a number of old and recent pavements that had either known design criteria or substantial evidence of good, fair, or poor performance were also examined. Figure 1 shows the existing DSMs for the areas tested.

Because a wide range of thicknesses of BC existed over most of the pavement and subsoil conditions, it was possible to plot DSM versus thickness of BC curves as shown in Figure 9. As may be noted from that figure, there is a similar curvature for each of the types of pavement. The intercepts on the DSM axis indicate the expected magnitude of the DSM on the several granular base and subsoil systems. These curves reveal increases in DSM of from 35 to 175 kN/cm of BC (20 to 100 kips/in of BC).

The next step was to calculate the required overlay by "standard" FAA methods. This entailed using equivalency factors to convert the existing sections to "conventional" flexible pavement sections and entering the CBR versus thickness curves developed for the project to determine the thickness of additional material required above each specific layer. A lower limiting CBR value of 2 percent was used for the very weak soils and an upper limit of 50 percent was used for the strongest subgrade. The required additional thicknesses of granular base, granular subbase, and granular fill were converted back to BC by using equivalency factors. The selected equivalency factors were based on evaluation of FAA standards and data in the Asphalt Institute Manual MS-11 (8).

The computed overlays were plotted against the DSM values for each of the specific test locations, and approximate curves were developed for one level of traffic. These curves were compared for shape and intercept values with the DSM versus thickness of BC curves. A final series of design DSMs and curves for each traffic level was selected from this comparison, and a typical set is given in Figure 10.

Figure 9. DSM versus bituminous concrete thickness.

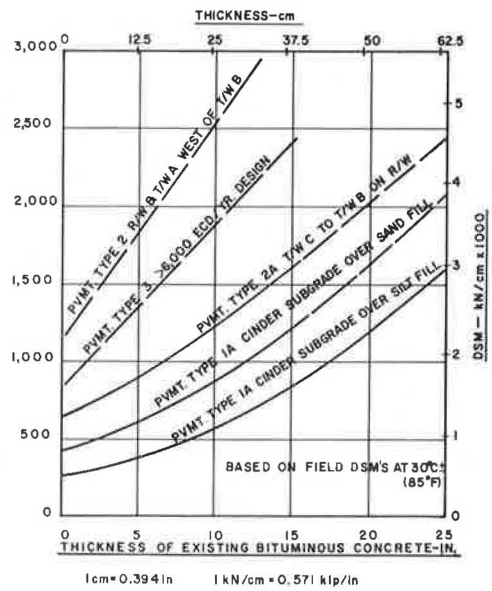
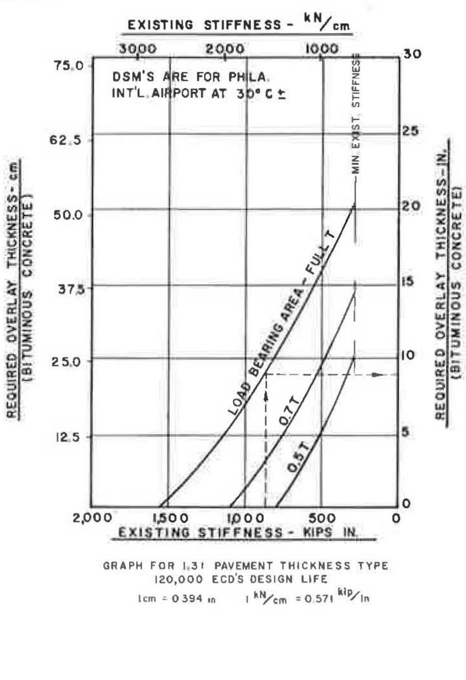


Figure 10. Typical DSM versus overlay thickness graph.



Selection of Recommended Overlays

To determine the overlay thicknesses to be recommended, we checked each analysis area by using average thickness, condition, and CBR data for the area. Overlay estimates were made by using the DSM versus overlay curves, the FAA equivalency method, and the Asphalt Institute MS-11 method (8) for design of new pavements.

Although the resulting three values were usually within 2.5 to 5 cm (1 to 2 in) of each other, there were a number of places where the thickness by DSM was several centimeters greater or less than by the other methods. The selected values were generally in conformance with the FAA method but were modified where the DSM or condition indicated a need. Minimum overlay thicknesses of FAA binder and surface requirements were recommended regardless of computed thicknesses because none of the existing BC was built to current specifications. Transition sections were recommended at certain locations, and the final overlay thicknesses were adjusted by the designers to achieve proper transverse and longitudinal grades.

CONCLUSIONS

Applicability of Dynamic Data

A review of the data and analysis from the 3 projects suggests four conclusions.

1. Data and correlations on bituminous pavements from different airports should not be too strictly compared unless all the data (deflections for stiffness, basin shape, and variable frequency) have been corrected to a uniform temperature and the subgrade conditions are similar.

2. A detailed knowledge of the construction history is necessary to develop an adequate investigation program. Knowledge of the current physical condition and strength of the pavement and soil layers is necessary to interpret the dynamic data. Special attention should be given to the soils below the upper portion of the subgrade that may never have been processed and that may be significantly weaker than the top of subgrade.

3. Dynamic testing with a heavy vibrator reveals ranges of strength far wider than would be expected from design and construction history.

4. The use of dynamic testing can limit shutdown time on a runway to 1 to 3 days (or nights) including time for destructive testing in boreholes depending on length, width, and variability of conditions.

Suitability of Stiffness Concept

The following comments apply primarily to DSM as measured by the WES 7257-kg (16-kip) vibrator and to the method of usage of the data on these projects, which were not research oriented.

Benefits and Advantages

1. The expression of strength by a single number is very attractive because it is derived from the overall pavement, base, and subgrade-in-depth system regardless of the thickness, compaction, and strength of the individual layers.
2. For a given set of design criteria (the FAA CBR versus thickness in this case), the DSM correlates reasonably well.
3. The potential ability to eliminate the cumbersome and questionably accurate material equivalency methodology will be a great asset.
4. Being able to perform several hundred tests per day and thereby obtain a significant number of tests in relatively small areas will assist the reliability of evaluation and design studies and will permit the detection and mapping of weaker areas.
5. The existing FAA procedures are applicable only to conventional rigid and flexible pavements, but current work by WES and correlations with destructive tests will enable analysis of stabilized base and composite pavements.

Disadvantages and Considerations for Improvement

1. The meaning of the DSM in more theoretical terms and in relation to actual aircraft loads and layered systems analyses is needed and has been under development by WES.
2. The shape of the deflection basin is not considered in the current FAA procedure but should be developed because work by others and by us indicates that it can be a significant factor.
3. Loading well into the straight-line portion of the load deflection graph may not be necessary, and the maximum dynamic load to be required should be defined as a function of the critical airplane gear load. This is important for the development of commercially available equipment and procedures.
4. The best frequency to use, or the need for frequency sweeps, should be firmly established. The variable frequency method is undergoing further investigation.
5. The current FAA data collection and analysis system does not permit determination of the thickness, condition, or strength of individual layers by nondestructive testing data alone. Further analysis of the curved portion of the load deflection plot and the deflection basins may be fruitful. Wave-velocity measurements are another approach. The significance of these determinations is in knowing where the controlling layers exist and being able to evaluate their effect on the pavement performance under future traffic. The ability to further minimize borings would also be helpful.

ACKNOWLEDGMENTS

We appreciate the cooperation of the Philadelphia Division of Aviation and their permission to use the data given here. Urban Engineers, Inc., of Philadelphia was the prime consultant.

The conclusions and opinions expressed are ours and do not necessarily represent those of our employers, the airport owners, or the project consultants.

REFERENCES

1. J. W. Hall. Nondestructive Testing of Pavements: Final Test Results and Evaluation Procedure. Air Force Weapons Laboratory, Technical Rept. AFWL-TR-72-151, June 1973.
2. J. L. Green and J. W. Hall. Nondestructive Vibratory Testing of Airport Pavements. Federal Aviation Administration, Vol. 1, Rept. FAA-RD-73-205-I, Sept. 1975.
3. J. W. Hall and D. R. Elsea. Small Aperture Testing for Airfield Pavement Evaluation. U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Miscellaneous Paper S-74-3, Feb. 1974.
4. Soil and Pavement Investigation for Improvements to Runway 9L-27R and Related Taxiways, Philadelphia International Airport. SITE Engineers, Inc., Sept. 18, 1974.
5. Use of Nondestructive Testing Devices in the Evaluation of Airport Pavements. Federal Aviation Administration, FAA Order 5370.7, Aug. 7, 1975.
6. Evaluation of Airfield Pavements Other than Army-Flexible Pavement Evaluation. Departments of the Army and the Air Force, Technical Manual 5-827-2 and Air Force Manual 88-24, Oct. 1968.
7. D. N. Brown and O. O. Thompson. Lateral Distribution of Aircraft Traffic. U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Miscellaneous Paper S-73-56, July 1973.
8. Manual MS-11. Asphalt Institute, College Park, Md.