Figure 2. Summary showing the relative amounts of each distress type present on Runway 1R-19L.

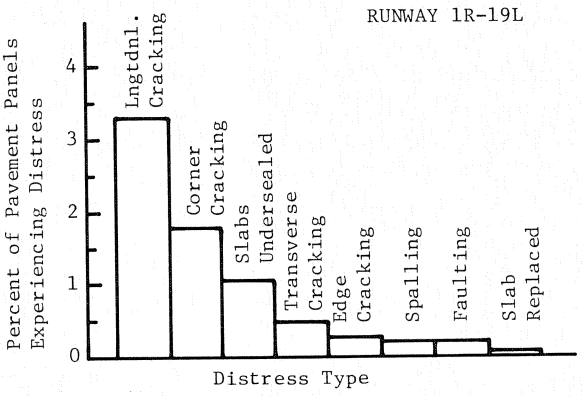


Figure 3. Summary of the longitudinal variation of distressed panels for Runway 1R-19L.

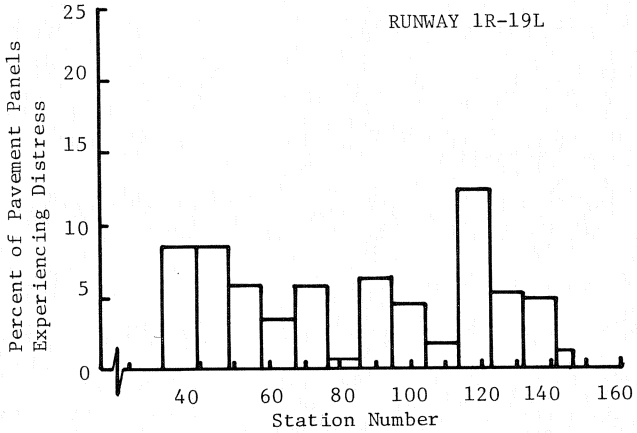
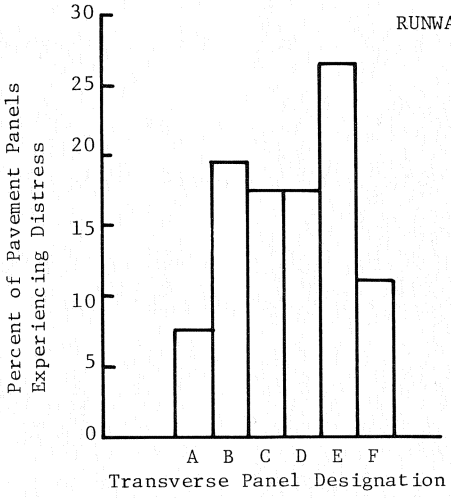


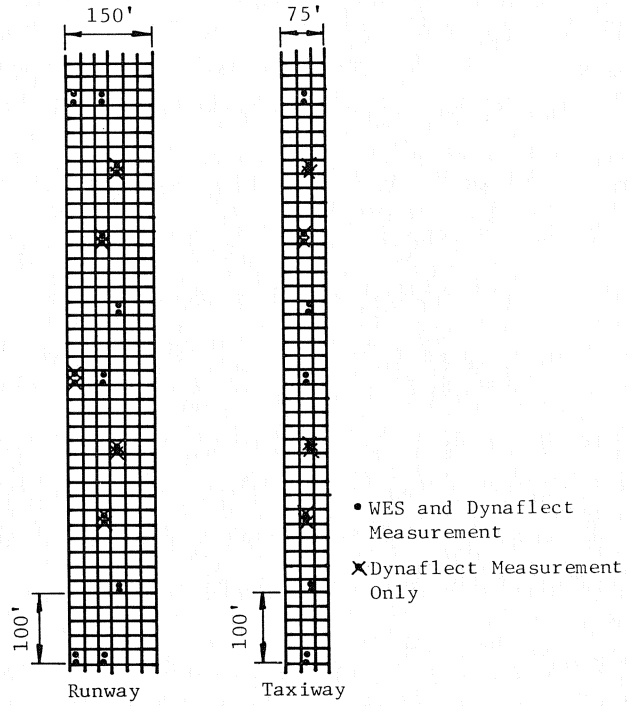
Figure 4. Summary of the transverse variation of distressed panels for Runway 1R-19L.



Nondestructive Deflection Testing

Nondestructive deflection tests were made on all of the pavements using both the heavy load vibrator from the Corps of Engineers Waterways Experiment Station in Vicksburg and the Dynaflect. The WES vibrator was intended to serve as the primary evaluation equipment, with the Dynaflect providing back-up as well as additional data. Approximately 1350 deflections were measured with the WES vibrator and 2700 with the Dynaflect. Readings were taken at both the joint and midspan of each panel measured. On the taxiways, edge and corner deflections were also measured. The typical measurement patterns used on runways and taxiways are shown in Figure 5.

Figure 5. Deflection measurement pattern used on on runways and taxiways.



Analysis of Deflection Data

Deflection profile plots were made for each lane of measurements on each runway and taxiway. The Dynaflect deflections were plotted by computer and the WES vibrator deflections, for a load of 4545 kg (10 kips), were then hand-plotted on the same axes. The deflection scale, however, was multiplied by a factor of 10 for the WES data. Examples of these plots are shown in Figures 6, 7, 8 and 9 for Runway 1R-19L. Plots are presented for left and right of centerline and for midspan and joints.

The WES vibrator deflections were analyzed to divide the slab-center deflection plots into different pavement sections according to the criteria in FAA publication AC 150/5370-11. These sections are shown on the deflection plots along with the mean deflections for the sections for both the WES and Dynaflect data. In cases where two sections of a runway or taxiway have means that are not signi-

Figure 6. Deflection profile plots for slab center deflections on Runway 1R-19L 3.66 m. (12 ft.) rt. $\mathbb{C}$L.*

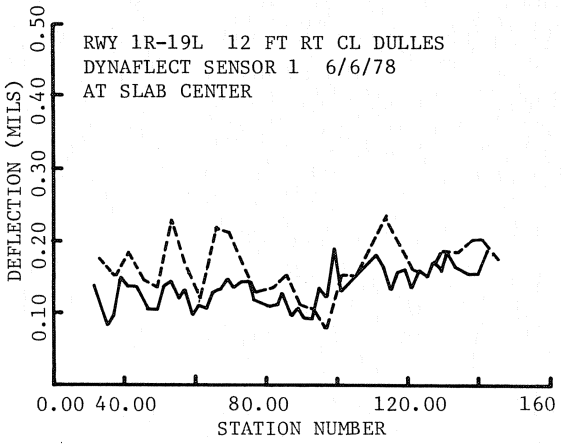


Figure 7. Deflection profile plots for joint deflections on Runway 1R-19L 3.66 m. (12 ft.) rt. $\mathbb{C}$L. *

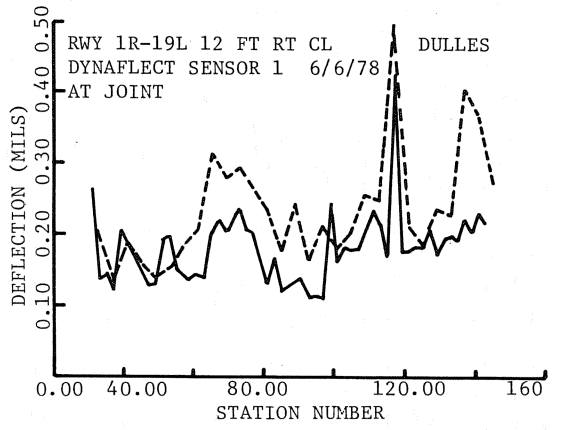


Figure 8. Deflection profile plots for slab center deflections on Runway 1R-19L 3.66 m. (12 ft.) lt. $\mathbb{C}$L.*

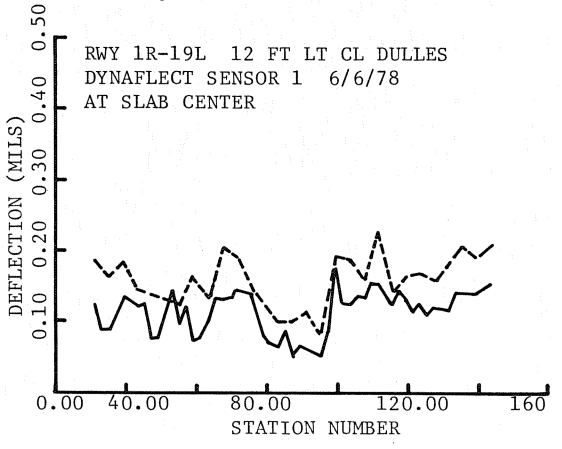
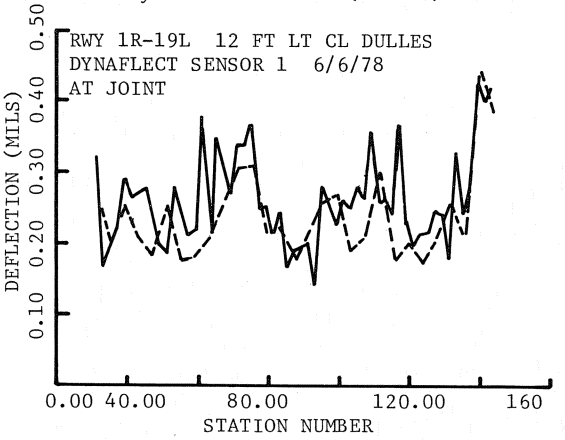


Figure 9. Deflection profile plots for joint deflections on Runway 1R-19L 3.66 m. (12 ft.) lt. $\mathbb{C}$L.*



* Note: ——— Dynaflect deflections (deflection scale as shown)
------ WES vibrator deflections (deflection scale is 10 times as shown)
1 mil = .0254 mm., 1 foot = 30.5 cm.

Table 1. Pavement sections and corresponding mean deflections.

Pavement Area	Stations	Mean Deflection, mils* WES Vibrator	Dynaflect	Pavement Area	Stations	Mean Deflection, mils* WES Vibrator	Dynaflect
Rwy 1R-19L	30 - 73	1.80	.13	Txwy W-2	14 - 32	1.43	.14
	73 - 105	1.20	.11		32 - 59	1.11	.11
	105 - 145	1.80	.13		59 - 87	1.42	.13
RWY 12-30	30 - 75	1.34	.12	Txwy E-2	102 - 113	1.40	.11
	75 - 103	1.82	.15		113 - 127	0.86	.09
	103 - 130	1.34	.12		127 - 145	1.40	.15
Rwy 1L-19R	30 - 70	1.49	.11	Txwy T-1	4 - 34	2.02	.10
	70 - 90	1.71	.13		34 - 44	2.66	.11
	90 - 145	1.49	.13	Txwy T-2	4 - 44	2.10	.10
Txwy E-1	30 - 85	1.68	.13				
	85 - 129	1.18	.11				
	129 - 137	3.50	.14	Jet Ramp R-1 Lane 1 (On Taxiway)	3 - 17	1.26	.10
Txwy W-1	14 - 145	1.57	.17		17 - 33	.82	.06
					33 - 44	1.26	.09
Txwy N-1	30 - 43	2.52	.19				
	43 - 87	1.62	.15				
	87 - 110	2.52	.20				
	110 - 131	1.62	.16				

* 1 mil = .0254 mm.

ficantly different, the values were combined to obtain one mean for both sections. The mean deflections for the major pavements are listed in Table 1.

Discussion of Deflection Analysis

The primary purpose of the deflection analysis was to make relative comparisons of the structural condition of the different pavement areas and then determine subgrade support values for each pavement section based on the deflection results. Most of the pavement areas have mean deflections within a relatively narrow range. Since all of the pavements have the same structural cross-section, i.e., 38.1 cm. (15 in.) of PCC on 22.9 cm. (9 in.) of granular base, any differences in deflection are due primarily to differences in subgrade support and also possibly to traffic usage.

However, it must be kept in mind that although differences in deflection between pavement sections were shown, the overall range of differences is small. Thus, it can be concluded that the deflection analysis showed no relative strong or weak areas.

Analysis of Joint Condition Using Deflection Data

The WES vibrator deflection data were used to evaluate the condition of joints on the three runways and their adjacent parallel taxiways. The ratio of joint deflection to midspan deflection for an individual slab is an indicator of the efficiency of performance of the joint. If the deflection measured at the joint and midspan of the slab are equal, i.e., the deflection ratio equals 1.0, the joint has ideal load transfer capacity. However, if the joint deflection is considerably greater than the midspan deflection, i.e., the deflection ratio is significantly greater than 1.0, say 2.0 or greater, the joint has lost some of its intended load transfer capacity, consequently the stresses are higher. Mean values of joint/midspan deflection ratios for the pavements analyzed are given in Table 2. The values range from 0.97 to 1.52 which indicates efficient joint performance.

Table 2. Ratio of joint deflection to midspan deflection.

Pavement Area	Mean Joint/Midspan Deflection Ratio
Runway 1R-19L	1.49
Runway 1L-19R	1.52
Runway 12-30	1.30
Taxiway E-1	1.49
Taxiway W-1	1.26
Taxiway N-1	.97

Analysis of Shoulder Deflections

Deflection measurements were made with both the WES vibrator and Dynaflect on the shoulders of the runways and their parallel taxiways. Measurements were alternated from right shoulder to left shoulder and spaced at 305 m. (1000 ft.) intervals for the vibrator and 152.5 m. (500 ft.) intervals for the Dynaflect. The vibrator deflections were measured at loads ranging from 2273 kg. to 4545 kg. (5 to 10 kips) rather than the constant 4545 kg. (10 kip) load used for the PCC pavement. This was done because of the fluctuating response on the thin shoulder pavement (5.1 cm. (2 in.) of asphaltic concrete). Mean deflections are presented in Table 3 for both the WES vibrator and Dynaflect deflections. The deflection magnitudes are approximately ten times greater than those measured on the PCC, which reflects the response to load of thick versus thin pavements.

Variation in deflection has little meaning with respect to the intended purpose of the shoulders, i.e. to serve as a cover for dust prevention and soil erosion due to wind and water.

Table 3. Deflection measurements on shoulders.

Pavement Shoulder	Mean Deflection, mils* WES Vibrator	Dynaflect
Runway 1R-19L	10.9	2.5
Runway 1L-19R	12.6	3.6
Runway 12-30	11.6	2.3
Taxiway E-1	12.4	3.4
Taxiway W-1	10.5	3.6
Taxiway N-1	12.8	2.5

*Note: 1 mil = .0254 mm

Comparison of WES Vibrator and Dynaflect Deflections

The deflection profile plots show both the WES vibrator and Dynaflect deflections plotted on the same axes. As previously stated, it must be kept in mind that the deflection scale for the WES vibrator is 10 times greater than that for the Dynaflect. The purpose of these plots was primarily to compare trends of deflection variation along the pavement, indicating that both devices induce very similar load response in the pavement, even though the load magnitude differs by a factor of 10 - 4545 kg. (10,000 lbs.) for the WES vibrator and 454.5 kg. (1000 lbs.) for the Dynaflect.

The correlation in deflection magnitude between the WES vibrator and Dynaflect was investigated in two ways - (1) plots of Dynaflect deflection versus WES vibrator deflection for measurements at the same specific locations, and (2) calculations of the ratio of WES deflection to Dynaflect deflection. An example plot is shown in Figure 10, for midspan deflections on Runway 1R-19L. Although the plot shows some scatter, there is a definite correlation trend between the two devices. The points are generally above the 10:1 slope line indicating a larger ratio would give a good correlation. Hence, the concept of stress sensitivity of the subsoil is illustrated.

Results of calculations of the ratio of WES vibrator deflections to Dynaflect deflections are presented in Table 4. Midspan and joint measurements are presented separately. Results show the

WES vibrator deflections to be 13.2 times greater than the Dynaflect deflections for the midspan measurements and 11.2 times greater for the joint measurements. Since these values are greater than the load ratio of 10:1, the soil stiffness decreases with increasing load as was shown by lab tests. This indicates that there is a differnece between the two devices for joint measurements and midspan measurements.

Figure 10. Plot of Dynaflect deflections vs. WES vibrator deflections.

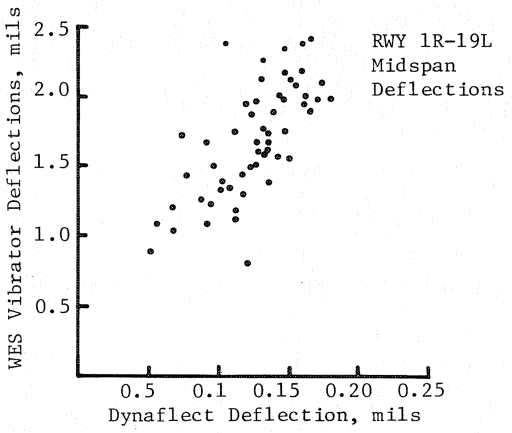


Table 4. Ratio of WES vibrator deflections to Dynaflect deflections.

Pavement Area	Average Ratio Midspan Measurements	Joint Measurements
Runway 1R-19L	13.78	11.23
Runway 1L-19R	13.58	11.03
Runway 12-30	12.28	11.38
Average	13.21	11.21

Derivation of Aircraft Operational Data

Several of the primary inputs to the pavement fatigue analysis were estimates of historical and future aircraft type, volumes, and operational patterns over Dulles airfield pavements. As a fundamental assumption, only larger aircraft types, 40,000 lbs. or more in weight, were considered. Smaller aircraft have widely varied taxi patterns and few exert any significant wheel loading pressures when compared to the larger aircraft types.

Weights for Design Aircraft

The fatigue analysis used to analyze the remaining life in the pavements considers the damage done to the pavement by each significant aircraft. Since each one has a different wheel load, they each cause a different stress in the pavement. The total damage resulting in the pavement is a sum of the damage done by each specific aircraft. The aircraft that have significant differences in landing and take-off weights are divided into two separate cases. For this analysis the Concorde, B747, and B707 were divided into separate landing and take-off weights. A single weight was used for each of the other aircraft. Table 5 presents the aircraft used in the analysis and the total weight, wheel load, tire pressure, and gear configuration of each.

Aircraft Operations

Since the fatigue damage on a section of pavement is a function of the number of aircraft loads the section has received, it was necessary to determine the number of each aircraft that has used each section of pavement in the past and the number projected for the future. The total number of cumulative aircraft of each type using the airport was determined for years 1978, 1981, 1985, 1990, and 1995. This data is given in Table 6. Each runway, taxiway, and ramp was divided into different pavement sections based on the deflection results (these are given in Section 3) and then further divided if necessary according to the variation in number of aircraft using the pavement. The total cumulative operations on the different sections were determined. These operations were divided into operations per aircraft type and then adjusted by traffic wander factors to account for transverse variation in wheel loads on the pavements. The final resulting number of operations for each aircraft was then input into the fatigue analysis to determine the remaining life and projected future end of useful life.

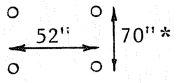
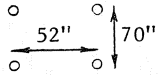
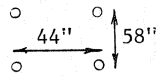
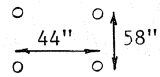
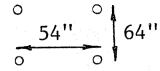
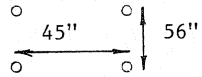
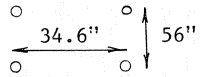
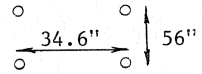
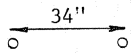
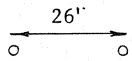
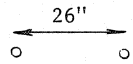
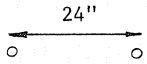
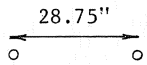
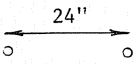
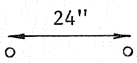
Materials Characterization

A total of 60 borings was made throughout the various pavement areas being evaluated. The locations of the borings were selected based on the results of deflection tests, and were chosen to represent areas of both low and high deflections. They were spaced at approximately 610 m. (2000 ft.) intervals on the runways and taxiways. Full depth core samples were taken from the PCC and the underlying granular base was augered and placed in bags. The subgrade was then sampled by pushing Shelby tubes to approximately 2.13 m. (7 ft.). Laboratory tests were run on each material in order to characterize them for input into the fatigue analysis.

Characterization of Portland Cement Concrete

The concrete cores were tested for tensile strength, compressive strength, modulus of elasticity, and density and also measured for thickness. The core thicknesses were very uniform, with the average being 38.1 cm. (15 in.), which was the design thickness. All of the strength tests indicate the concrete is of very good quality. The kPa (5.4×10^6 psi) was selected for input into the fatigue analysis.

Table 5. Aircraft weight data used in pavement analysis.

Aircraft Type	Total Weight (lbs)*	Wheel Load (lbs)	Tire Pressure (psi)*	Gear Configuration
Concorde (take-offs)	400,000	47,500	180	
Concorde (landings)	245,000	29,095	180	
B747 (take-offs)	610,000	36,219	204	
B747 (landings)	564,000	33,488	204	
DC-10	380,000	45,125	175	
New 2-Engine Widebody (B767)	270,000	32,063	175	
B707 (take-offs)	270,000	32,063	180	
B707 (landings)	210,000	24,938	180	
B727	150,000	35,625	168	
DC-9	90,000	21,375	163	
4-Engine Turbo-prop (L-188)	105,000	24,938	138	
Twin-Engine Turboprop (CV-580)	50,000	11,875	65	
4-Engine Piston (DC-6)	100,000	23,750	98	
2 Engine Piston Piston (340/440)	45,000	10,688	65	
Large GA (CV 580)	50,000	11,875	65	
Mobile Lounge (Plane-Mate) Empty	57,500	14,375	75	o
Mobile Lounge (Plane-Mate) Full	84,000	21,000	75	o

*1 lb = .45 kg, 1 psi = 6.895 kPa, 1 in = 2.54 cm

Table 6. Total aircraft operations on airport, cumulative from 1962.

	1978	1981	1985	1990	1995
SST	1527	4576	10208	21373	38313
747	28668	55858	103822	191140	312338
DC-10	25850	47300	99275	224015	421905
WB	0	0	0	34760	173470
707	424353	505863	623612	708367	798677
727	221936	284900	378840	543180	805420
DC-9	115874	122540	180235	274175	345015
4ET	17766	20301	26248	33893	39613
4EP	14520	14520	14520	14520	14520
2ET	20220	24101	29957	37575	45055
2EP	2266	2266	2266	2266	2266
Large GA	11915	15466	21190	28890	36590
Mobile Lounge	2852260	3624973	4860104	7301536	10510531

Characterization of Granular Base

Since it was not possible to obtain undisturbed samples of the base material, samples were recompacted at the moisture and density specified for construction and tested for modulus of elasticity. The average results were as follows:

1. Moisture content - 6%
2. Dry density - 2395 kg/m^3 (149.5 pcf)
3. Modulus of elasticity - 68,950 kPa (10,000 psi)

A modulus of 68,950 kPa (10,000 psi) was selected for input into the fatigue analysis.

Characterization of Subgrade

The subgrade samples were tested to determine modulus of elasticity, moisture content, dry density, gradation, and plasticity index. They were classified according to the FAA soil classification system and found to be predominantly E-7 soils. The overall average PI, moisture content, and dry density are as follows:

1. PI - 17
2. Moisture content - 24.2%
3. Dry density - 1666 kg/m^3 (104 pcf)

For determination of modulus of elasticity values to input into the fatigue analysis, each of the different pavement sections established by the deflection analysis was considered separately. Modulus values were established based on the deflections, laboratory tests of modulus versus deviator stress, and the stress condition resulting in the subgrade from each different aircraft load. Both WES vibrator and Dynaflect deflections were used to characterize the subgrade. The results for Runway 1R-19L are shown in Table 7. The subgrade modulus values were input into the fatigue analysis for computation of the stress in each pavement section for each aircraft load.

Comparison of WES Vibrator and Dynaflect Based on Subgrade Characterization

To compare the results obtained, the percent difference between the moduli determined for each device was calculated. The moduli determined by the WES vibrator data were used as the base for calculating the percent difference. The results are shown for each pavement section in Table 8.

As seen from the difference in Table 8, the Dynaflect data characterized the modulus higher than the WES vibrator data for approximately one-half the pavement sections and lower than the WES vibrator data for the other one-half. The closest comparison is for Runway 12-30, stations 103-130, for which the modulus values are equal.

Although these differences in moduli may appear high, they are not as significant as might be thought in their effect on the final result of the evaluation, i.e., computation of remaining life. The subgrade modulus value is not a highly significant factor (within a reasonable range) in computation of the stress in the PCC, which is a highly significant factor in determining the amount of remaining life.

Table 7. Characterization of subgrade for RWY 1R-19L.

	Modulus of Elasticity for Subgrade, psi*					
	STA. 30-73		STA. 73-105		STA. 105-145	
	WES Vibrator	Dynaflect	WES Vibrator	Dynaflect	WES Vibrator	Dynaflect
Concorde - T	14698	16387	22221	16572	14698	16387
Concorde - L	18785	21084	30497	22258	18785	21084
B747 - T	17723	19892	28500	20798	17723	19892
B747 - L	18394	20667	29924	21759	18394	20667
DC - 10 - 10	16902	18947	26807	19640	16902	18947
B707 - T	17987	20167	28842	21122	17987	20167
B707 - L	20207	22733	33613	24342	20207	22733
B727	20578	23066	33515	24684	20578	23066
DC - 9	24751	27864	42523	30893	24751	27864
B767	18809	21130	30685	22327	18809	21130
L-188	23338	26234	39381	28747	23338	26234
CV580	30038	33989	54921	39259	30038	33989
DC - 6	23850	26822	40493	29515	23850	26822
CV 340	30990	35097	57274	40825	30990	35097

* 1 psi = 6.895 kPa

Table 8. Comparison of subgrade characterization by WES vibrator and Dynaflect.

Pavement	Stations	Variation in Dynaflect Modulus Expressed as a Percent of WES Vibrator Modulus
1R-19L	30- 73	+12%
1R-19L	73-105	-27%
1R-19L	105-145	+12%
1L-19R	30- 70	+55%
1L-19R	70- 90	+23%
1L-19R	90-145	-25%
12-30	30- 75	-31%
12-30	75-103	+12%
12-30	103-130	0%
E-1	30- 85	+16%
E-1	85-129	-37%
E-1	129-145	+150%
W-1	14-145	-31%
N-1	30- 43	+14%
N-1	43- 87	+12%
N-1	87-110	+30%
N-1	110-131	-18%

Determination of Remaining Life and Projection of Future Life

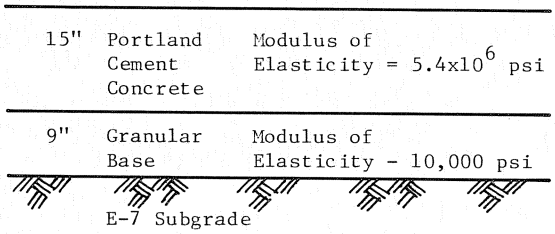
Determination of the remaining life in the pavements and projection of future life is based on computation of the damage caused by each individual aircraft and then summing the results to obtain the total damage. Different pavement sections throughout the airport have different amounts of remaining life, depending on the subgrade characterization, the aircraft types that have used the pavement section, and the number of aircraft that have used the section.

The pavement thickness is constant for all of the sections analyzed, so one basic cross-section was established for each section, except for the subgrade modulus which varies as previously explained. The cross-section used for the analyses is shown in Figure 11. The stress in the PCC was computed for each pavement section for each aircraft that uses the section and input into the fatigue model to determine the allowable number of each aircraft until a failure point is reached. This point is defined as a condition where approximately 10% of the slabs will experience extensive fatigue cracking. The pavement may still serve traffic at this point, but the roughness will increase rapidly. Furthermore, the structural integrity of the investment will be lost. The allowable number of applications, along with the actual number of applications were input into the damage model to determine the amount of life remaining in 1978 and the projected life remaining in 1981, 1985, 1990, and 1995. Plots of remaining life versus time were made for each pavement section.

The curves were projected to determine the anticipated end of the structural life of the pavement. The plots for Runway 1L-19R are shown in Figures 12, 13, 14, and 15 as examples of the method used. The analysis was done for both WES vibrator and Dynaflect data for the runways and major taxiways. WES vibrator data only was used for the other pavement sections. These values, however, represent only the fatigue cracking damage done by repetitions of aircraft wheel loads. There are other factors that have significant effects on pavement life and future rehabilitation planning. Some of these factors are:

1. The environment has significant detrimental effects on the pavement which accelerates its deterioration.
2. Rigid pavements deteriorate fairly rapidly after reaching a certain point - this cannot be fully accounted for in a fatigue analysis alone.
3. The PCC stress values used in the fatigue analysis are representative of the stress in the interior portion of the slab. In reality the most damaging and critical stresses occur at the joints, thus causing them to deteriorate and require rehabilitation sooner than the interior of the slab.
4. It is necessary to rehabilitate a pavement prior to the end of its structural life, because the time and cost of maintenance will become prohibitive before the final structural failure is reached. Consideration should be given to rehabilitating a pavement when the calculated remaining structural life reaches 10% to 15%.
5. The calculated remaining structural life must be tempered with engineering judgement. It is highly improbable that an exact year of the failure can ever be accurately predicted due to uncontrolled variables such as environment, materials properties, traffic patterns, and projected aircraft type.

Figure 11. Basic cross-section established for the remaining life analyses.



Modulus of elasticity varies with deflection measurement, aircraft type, and laboratory tests.

Figure 12. Remaining life vs. time for Runway 1L-19R, Stations 70-90.

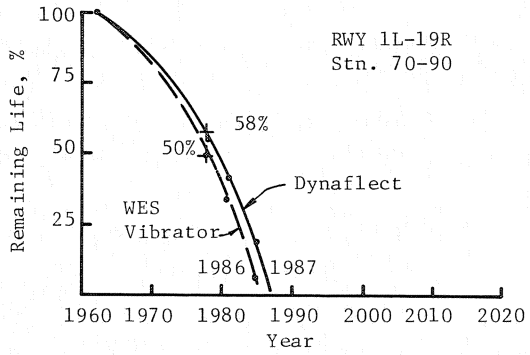


Figure 13. Remaining life vs. time for Runway 1L-19R, Stations 30-70.

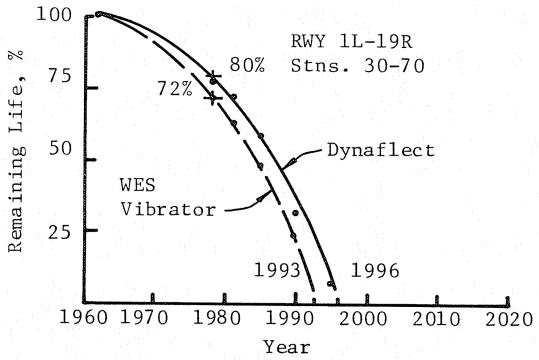


Figure 14. Remaining life vs. time for Runway 1L-19R, Stations 90-110.

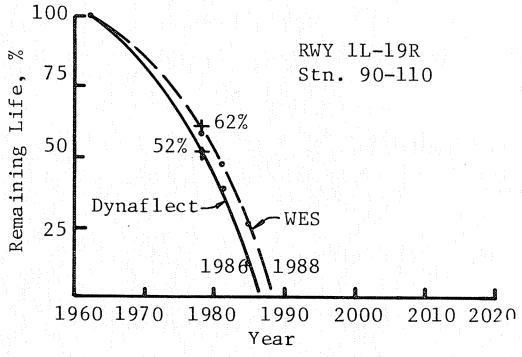


Figure 15. Remaining life vs. time for Runway 1L-19R Stations 110-145.

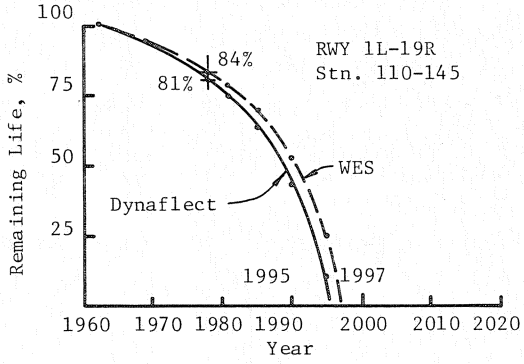
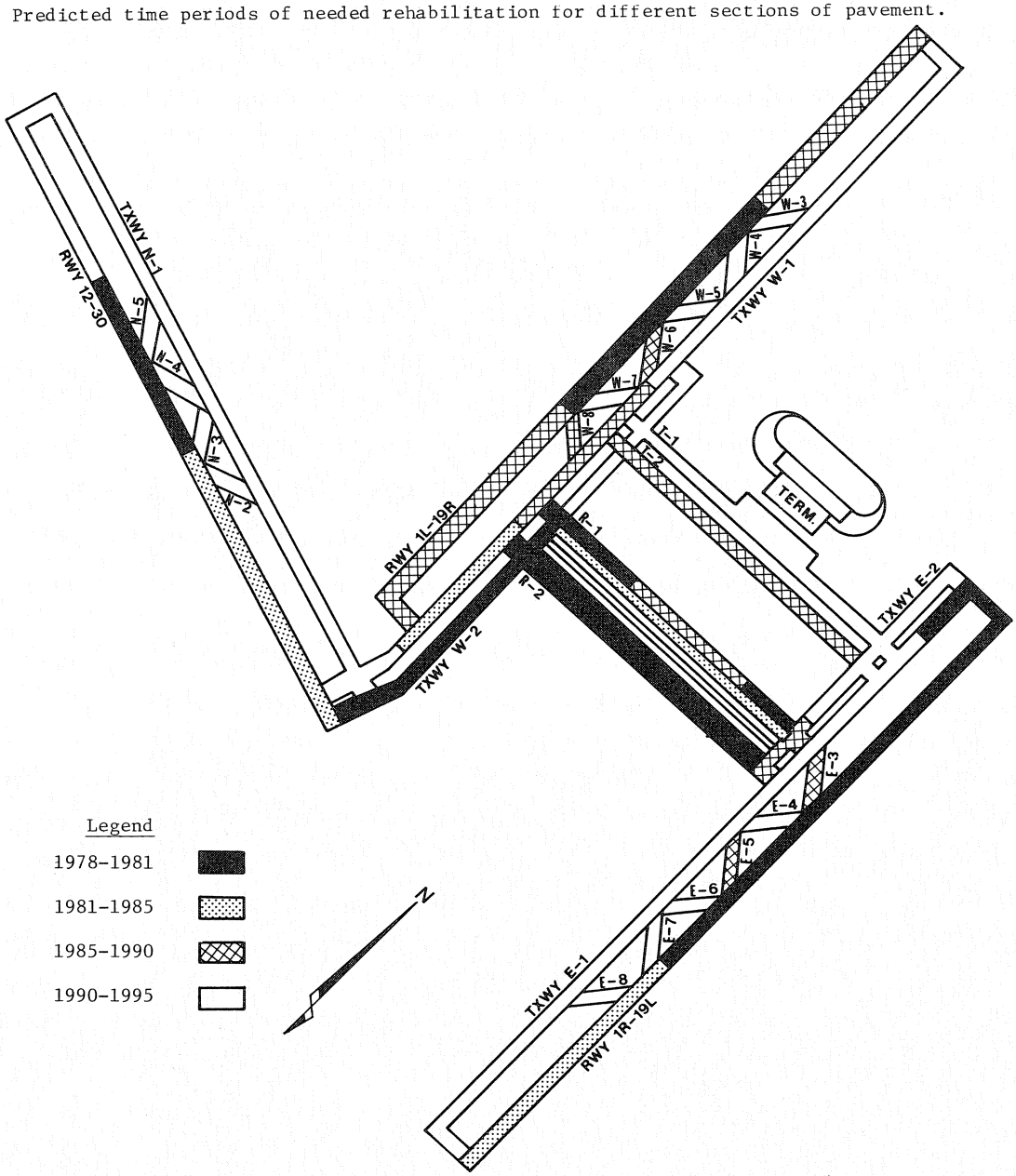


Figure 16. Predicted time periods of needed rehabilitation for different sections of pavement.

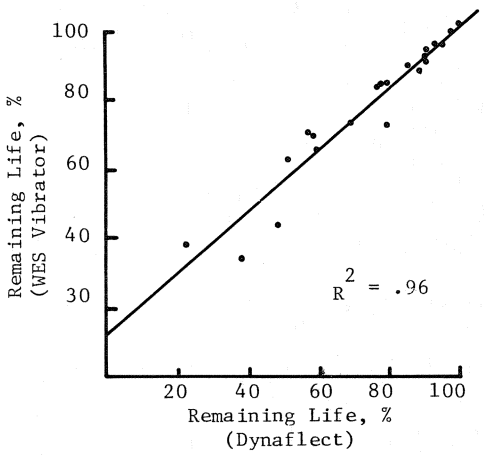


Based on the above reasons along with the structural fatigue analysis the pavement sections were grouped into time periods based on when they can be expected to need major maintenance or substantial rehabilitation, either by removal and replacement of slabs or by overlaying. The results are shown in Figure 16. Although there may be some deviations from the classifications by a few specific sections, the groupings give a good relative comparison between sections of anticipated rehabilitation needs. This information can serve as an aid in short range and long range funds programming.

Comparison of WES Vibrator and Dynaflect Based on Remaining Life Calculations

The WES vibrator and Dynaflect were compared previously based on deflections and subgrade characterization. A comparison based on remaining life predictions may be a more meaningful comparison than the first two, since remaining life is the final product of the pavement evaluation. Review of the remaining life values shows them to compare very closely. The values were plotted against each other in Figure 17 to graphically illustrate the correlation. The line of best fit is shown along with the correlation coefficient, R^2, which is 0.96. These results indicate a very good comparison between the WES vibrator and Dynaflect results.

Figure 17. Comparison of WES predictions vs. Dynaflect for remaining life.



Conclusions and Recommendations

Significantly, the principal study conclusion is that the operational airfield pavements at Dulles International Airport are in good condition. Most of the limited amount of distress now present is a result of environmental conditions, primarily temperature changes which have caused slab cracking, and freeze-thaw cycles which have resulted in random joint and corner problems. Some of the environmental distress has been aggravated by repetitive application of aircraft wheel loads. The pavement life projections described are based on prediction of the occurrence of fatigue cracking due to repetitive wheel load applications. Effects of this load-associated damage can be delayed to some extent by repairing the environmental distress that is now present.

Recommendations for Immediate Pavement Rehabilitation

Based on a review of the condition survey data, the following rehabilitation and maintenance measures were recommended for the next one to two years:

1. Remove and replace the airfield pavement panels that are significantly distressed.
2. Rehabilitate the asphaltic concrete shoulder pavement.
3. Seal existing cracks and repair spalled areas.

A total of 117 panels were recommended for removal and replacement.

Recommended Long Range Alternative for Programming Purposes

Evaluation of alternatives in order to select the optimum one for fund programming requires a comparison between overlays versus removal and replacement. Since an overlay will result in a difference in elevation between the overlaid pavement and adjacent pavement areas, complete sections of the airport must be overlaid at one time, i.e., a runway and its complete taxiway system. Obviously, there would still need to be grade transitions at some of the intersections, particularly near the terminal aprons. However, the overlay method has the significant advantage of utilizing the remaining strength of the existing pavement. This benefit is lost when the pavement is removed and replaced. Conversely, removal and replacement will allow smaller sections of pavement to be constructed at one time reducing the area of closure during construction. This procedure also eliminates any problems that would result from grade differences and transition areas. However, a greater thickness would be required for replacement than would be needed with an overlay significantly increasing the cost. There is also additional cost associated with the removal of the existing pavement.

In summary, the following long range plan was recommended:

Spot replacement of damaged panels over the next 5 to 6 years and beginning of an overlay program in the mid-1980s. With this scheme the overlay schedule would be:

1. Runway 1R-19L and adjacent taxiways - 1988
2. Runway 1L-19R and adjacent taxiways - 1989
3. Jet ramps and adjacent taxiways - 1990
4. Runway 12-30 and adjacent taxiways - 1991
5. All other areas - 1992

Pavement Monitoring Program

The ideal approach to programming the overlay construction would be to establish a pavement monitoring program to study the condition of the pavements with time. Using this approach will allow the best planning to be made, from both a pavement structural and economic viewpoint.

Recommendations for the pavement monitoring program are:

1. Make annual condition surveys of all airfield pavements and compare to the previous surveys. Rapid deterioration can be detected by such annual surveys. These areas can then be scheduled for early rehabilitation/replacement.
2. Perform nondestructive deflection testing on all the pavements every few years. This will allow determination of the basic structural capacity of the pavements for comparison with previous test results. These periodic deflection testing programs would not need to be as extensive as the program just completed, since a large data base has now been established.
3. Determine the traffic patterns and volumes of usage for the same time frame and compare the results to the traffic projections.
4. Review all data collected during periodic testing to ascertain whether significant changes are taking place. If so, additional projections of remaining useful lifespan would be made so that the rehabilitation effort could be reprogrammed in response to changing circumstances.

The monitoring program as outlined is the ideal way to achieve the highest confidence level for rehabilitation planning. It would also insure maximum utilization of the investment in the origianl airfield pavements.

Reference

1. ARE Inc - Engineering Consultants and Howard Needles Tammen and Bergendoff. Evaluation of Airfield Pavements at Dulles International Airport. Report No. HN-10, September 1979.