Using Nondestructive Testing in the Semi-Arid Zone of Peru

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Presented in this paper is a methodology for nondestructive pavement testing and its application in the rehabilitation program of the Talara and Piura Airports, located in the Pacific region of Peru approximately 1000 to 1100 km north of Lima. The area's water table is 2 to 8 m under the subgrade surface and the annual rainfall is less than 150 mm. Each airport has a 2500-m runway and is used mainly by medium-sized and small jet aircraft such as B-727s, B-737s, and F-28s. The existing pavement materials do not meet standard specifications for plasticity, gradation, and California bearing ratio, and the thickness is less than that recommended by the Federal Aviation Administration. Nevertheless, 15 years' experience generally indicates adequate pavement performance. It was concluded that the most practical way to interpret the actual performance of these marginal materials is to use nondestructive testing to determine the elastic parameters of the existing subgrade and pavement. The methodology used in Piura and Talara is based on the Hogg model of a thin plate on an elastic foundation. The subgrade modulus can be determined without prior knowledge of the thickness or the characteristics of the pavement layers. The pavement modulus can then be calculated for any given load and center deflection. The survey indicates that existing pavement materials in such arid zones can generate sufficient bearing capacity to support traffic loads of B-727s, B-737s, F-28s, and DC-8s. The test results were used to upgrade the existing airport so that they could carry 600 annual operations of B-727s with total gross weights of 160,000 lb for 20 years.

During 1985 and 1986, the Peruvian Air Transportation Authority started the rehabilitation program of Piura and Talara Airports. Both are located in the northern Pacific region of the country about 1000 to 1100 km north of the capital, Lima. These airports are mainly used inter alia as alternatives to the Pan-American Highway connecting the northern Pacific areas with the capital, because some sections of the highway are presently in poor condition. These airports are used mainly by medium-sized and small jet aircraft such as B-727s, B-737s, and F-28s. Occasionally, heavier aircraft, such as DC-8s, use Talara Airport. In both airports, the length of the runway is about 2500 m. Piura Airport is located at coordinates 05°12'S, 8°37'W and its elevation is 35 m above sea level. The airport pavement suffered severe failure in 1982 because of a combination of flooding—very rare in this area—and pavement overstress. The 1982 flash flooding was unusual with a return period of over 100 yr. This pavement failure has limited the effective length of the runway to 1700 m until completion of rehabilitation in November of 1986.

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The Talara Airport is located about 150 km north of Piura and is used by passengers and to transport petroleum drilling and production equipment to the local on- and offshore oil fields. In addition, Talara Airport is also used as an emergency airport in case of bad weather in Lima or Guayaquil, Ecuador. Standard testing shows that the existing pavement materials do not meet FAA specifications for plasticity, gradation, and strength. Nevertheless, over 15 yr of experience have shown that even these marginal materials have performed adequately in the local arid conditions, with annual rainfall less than 150 mm and the water table 2 to 8 m under the subgrade surface. It was found that the most practical way to interpret the actual performance of the substandard material is to use nondestructive testing (NDT) to determine the in situ elastic parameters of the existing subgrade and pavement materials. NDT was used to minimize the cost of upgrading both airports.

METHODOLOGY

Nondestructive Testing

A rational methodology of pavement evaluation should be independent of the type of equipment used to test the materials. Although pavement materials are nonuniform and nonisotropic, and although their stress-strain relationships are not linear, one would expect to reach similar results using different NDT devices. Such a methodology has been successfully implemented worldwide *(1–4)*. Figure 1 presents a typical comparison of layer moduli for five different types of airport pavement tested with (a) the pavement profiler (PP), (b) a 16-kip vibrator (*V*), and (c) the falling weight deflectometer (FWD) *(4)*. Each NDT device was operated by a different operator on a different day and at randomly selected testing points for each one of the testing areas. The same subgrade modulus was obtained from the different NDT devices. Similar conclusions were obtained for the pavement modulus [see Figure 1 *(4)*]. Test Area 1 was 20 in. of portland cement concrete (PCC). Test Areas 2 and 3 were flexible pavement with 11.0 and 5.5 in. of asphalt concrete (AC), Test Area 4 was a composite section with 7 in. of AC on top of 6 in. of PCC, and Test Area 4 was 10.5 in. of PCC.

Dynamic NDT devices, such as the PP, FWD, or even the dynaflect are not available in Third World countries like Peru. In such countries the Benkelman beam is used for structural pavement evaluation. The rebound deflection basin is obtained under a dual-wheel axle load and is interpreted by a pocket computer with 8K of random access memory (RAM) to determine the elastic moduli of the subgrade and pavement.

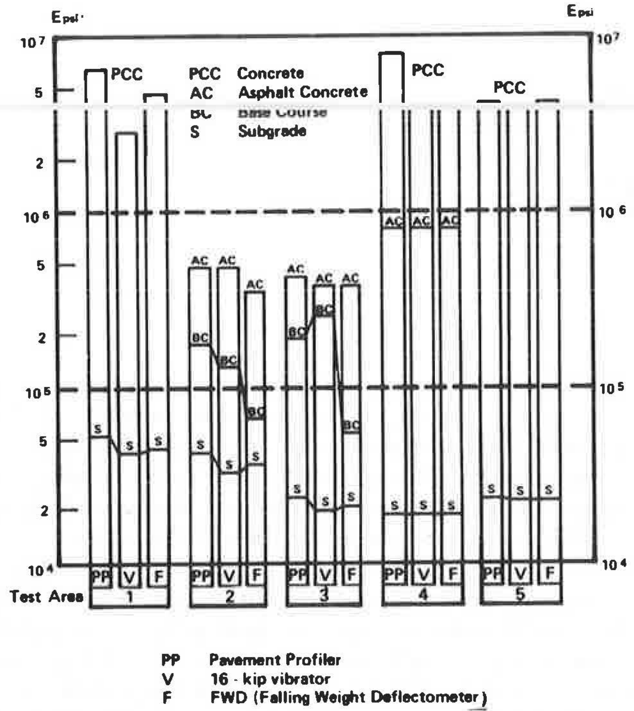

FIGURE 1 Comparison of lay moduli for five airport test areas determined by different NDT devices.

Demonstrated in Figure 2 is the use of the Benkelman beam for the rehabilitation study of Piura Airport. As shown in this figure, the beam operates on the lever principle. Every vertical movement of the tip of the beam generates a rotation of the beam through the pivot. A proportion of the tip movement is read with the dial gauge installed at the far end of the beam. The ratio of the rotating lengths of the beam is generally 1:4 (including the beam used in Piura); thus, the dial gauge at the end of the beam moves one-fourth of the vertical movement at the tip of the beam. Often the dial gauge is already calibrated to read the full tip movement (i.e., no multiplication by 4 is required).

The truck used in Peru had a single dual-wheel rear axle load (PP) weighing 8200 kg and a tire pressure of 4.9 kg/cm^2. Any value of PP can be used to determine the elastic parameters *(3)*. The truck moved away from the testing point at creep speed, and the rebound deflections were measured. This method was used to measure not only the maximum deflection under the rear axle ($D\phi$), but also to measure two additional deflections—D_{40} and D_{70}—at 40 and 70 cm away from the maximum, respectively. This nonroutine procedure was used to characterize the whole deflection basin needed in the structural evaluation methodology explained in the following sections. The choice of 40 and 70 cm was not arbitrary. The goal was to choose a distance at which the deflection would be about 50 percent of the maximum deflection. With little practice, it is possible to measure the offset deflections without having to stop the moving truck. A team composed of the truck driver and his assistant and an experienced engineer and his assistant was able to measure about 150 deflection basins in a typical working day.

Subgrade Modulus

In addition to being insensitive to the types of equipment used, the NDT methodology *(1–4)* has another advantage in that the subgrade modulus ($E\phi$) may be computed from the deflection bowl measurements without prior knowledge of the thickness or other characteristics of the pavement layers above the subgrade. The influence subgrade thickness is determined uniquely from the measured deflection basin *(1–4)*. This characteristic is extremely significant when pavement thickness is nonuniform, as in the case of Piura and Talara Airports. The Hogg model *(5, 6)* of a thin plate on an elastic foundation has been used to determine the subgrade modulus in the Peruvian airport projects. Extensive use of the Hogg model *(1–4, 7–9)* has shown it to yield satisfactory results for the modulus of elasticity of the subgrade ($E\phi$) or subgrade California bearing ratio (CBR) compared with values obtained from in situ testing. Figure 3, for example, presents the relationship between the measured and calculated deflection basin of flexible, rigid, and composite airport pavements *(4)*. This figure indicates agreement between the theoretical and the measured deflection basin. Another verification related to the subgrade strength is presented in Figures 4 and 5. These figures are taken from an NDT pavement evaluation study carried out in Thailand in 1984. Figures 4 and 5 indicate agreement between the in situ and the calculated CBR. The calculated subgrade CBR was determined by the following equation *(3, 9–12)*:

$$\text{CBR} = E\phi \text{ (in kg/cm}^2\text{)}/CE \qquad (1)$$

where *CE* is an empirical factor that varies mainly between 100 and 150 for in situ CBR between 2 and 30. *CE* = 130 is usually used for pavement-strengthening design *(3, 9)*.

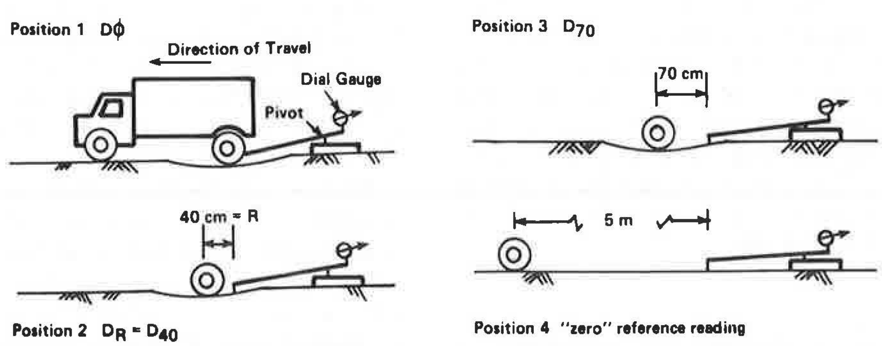

FIGURE 2 Schematic of Benkelman beam deflection procedure.

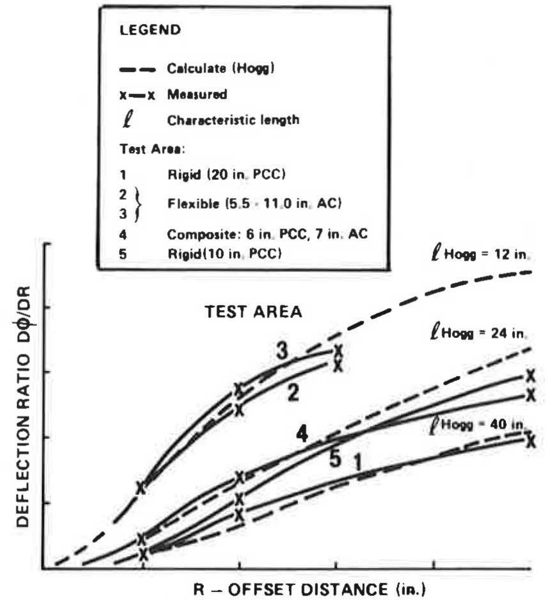

FIGURE 3 Comparison between measured and calculated deflection basin.

The use of the Hogg model for determining the subgrade modulus ($E\phi$) from the measured center deflection $D\phi$ and the deflection DR at an offset distance R are described in detail by Greenstein; Wiseman and Greenstein; Bergen and Greenstein; and Wiseman et al. *(1–4)*.

Determination of Pavement Modulus (E^*)

The combined modulus E^* of the AC and the base layers with a combined thickness of $HC = H_1 + H_2$ (see Figure 6) is determined by using the Odemark-Ullidtz equations *(13)* for equivalent thickness *(14)*. The equivalent thickness is determined according to the following equation:

$$HE = 0.9\,HC\,(E^*/E\phi)^{1/3} \tag{2}$$

The relationship between the center deflection $D\phi$ (between the dual wheels), the elastic modulus of the subgrade $E\phi$, and the pavement E^* is given in the following equation:

$$\begin{aligned} D\phi = {} & [(1+\mu)(PP)/2\pi]\,[(1/E^*)\,([2(1-\mu/r)] \\ & - [1/R(1)]\,\{2(1-\mu) + [Z(1)/R(1)]^2\}) \\ & + (1/E\phi)\,([1/R(2)]\,\{2(1-\mu) + [Z(2)/R(2)]^2\} \\ & - [1/R(3)]\,\{2(1-\mu) + [Z(3)/R(3)]^2\})] \end{aligned} \tag{3}$$

where

$$\begin{aligned} r &= 1.5A = 1.5a, && (4)\\ Z(1) &= HC + 0.6\,(A^2)/HC, && (4a)\\ R(1) &= \{[Z(1)]^2 + (1.5A)^2\}^{1/2}, && (4b)\\ Z(2) &= HE + 0.6\,(A^2)/HE, && (4c)\\ HE &= 0.9HC\,(E^*/E\phi)^{1/3} && (4d)\\ R(2) &= \{[Z(2)]^2 + (1.5\,A)^2\}^{1/2} && (4e)\\ Z(3) &= (HE + NI) + 0.6\,(A^2)/(HE + NI), && (4f) \end{aligned}$$

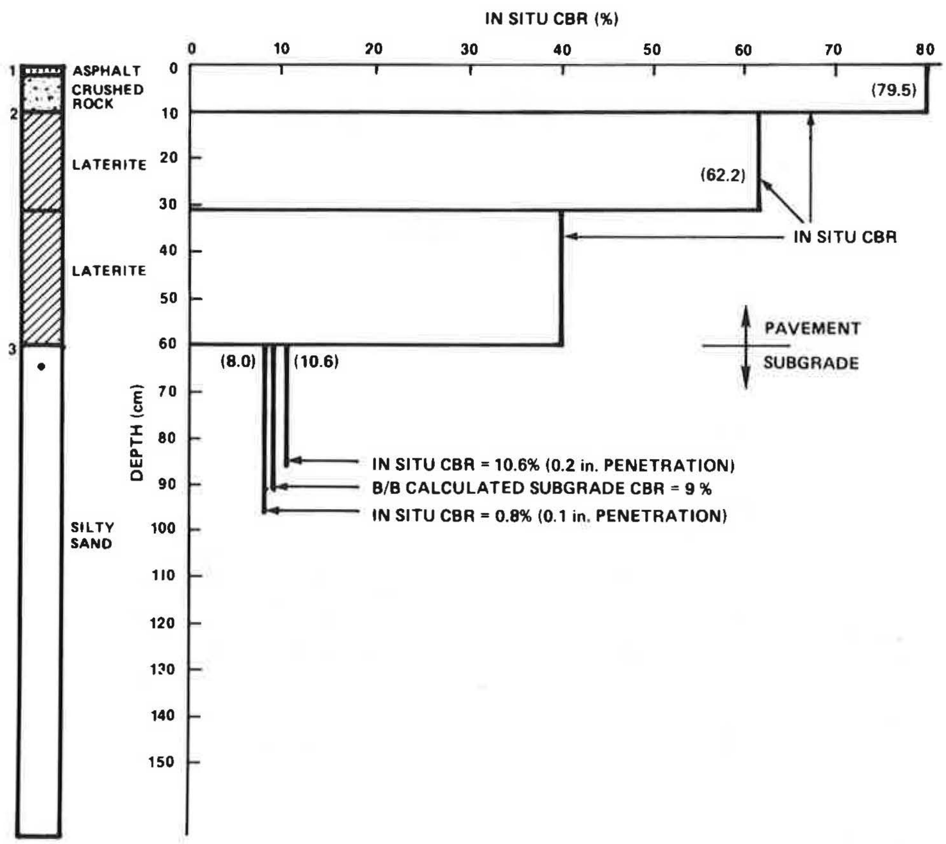

FIGURE 4 Relationship between calculated and in situ CBR (Thailand).

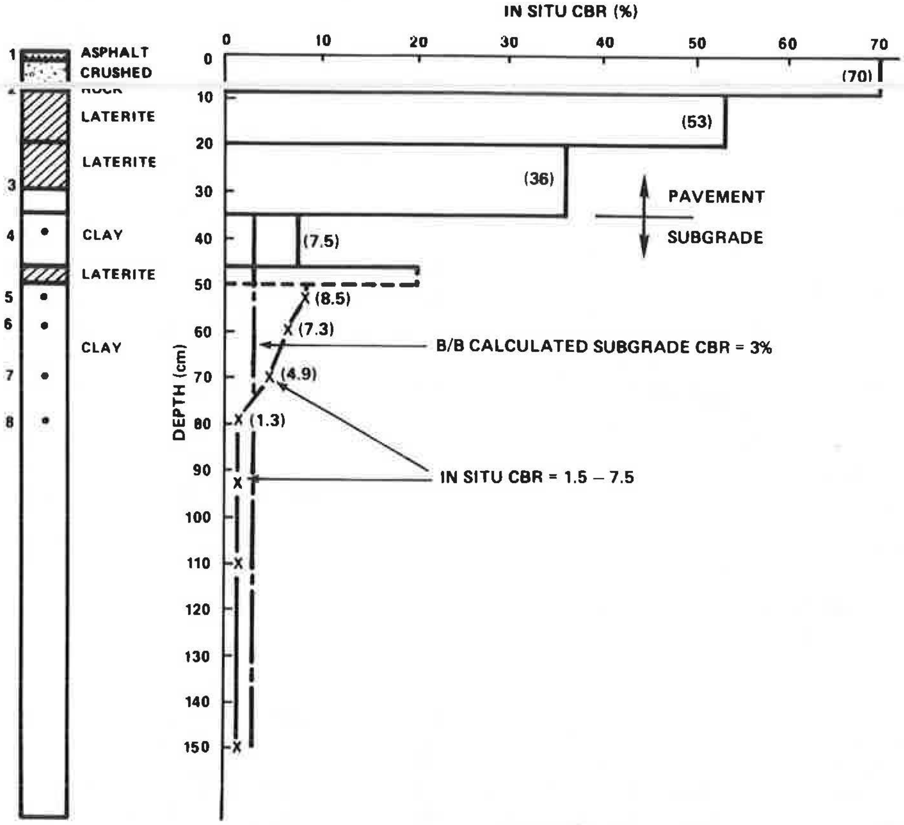

FIGURE 5 Relationship between calculated and in situ CBR (Thailand).

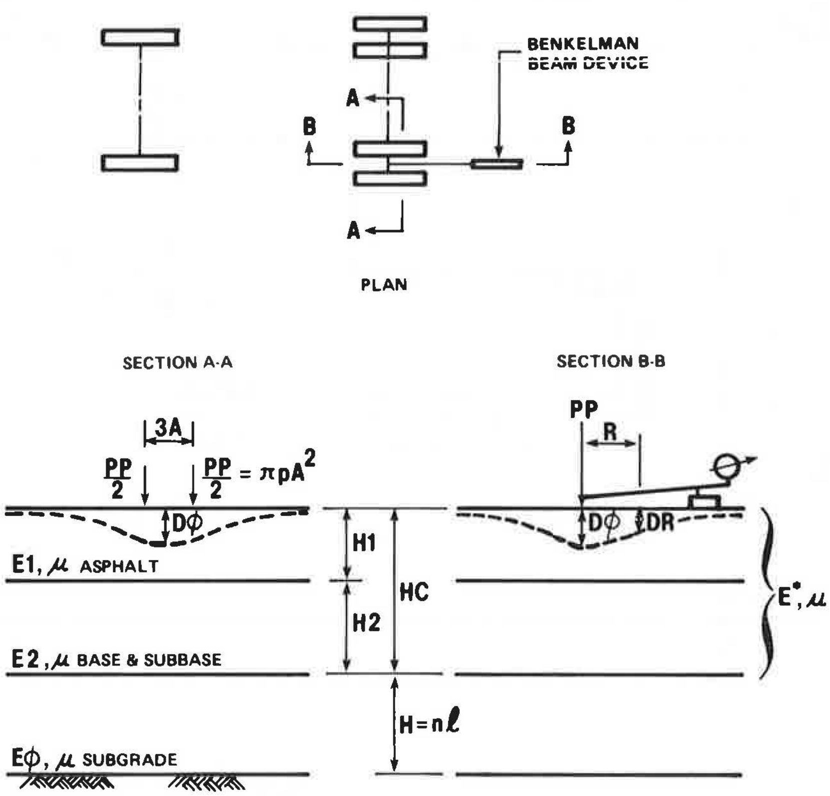

FIGURE 6 Subgrade pavement moduli parameters.

$$R(3) = \{[Z(3)]^2 + (1.5A)^2\}^{1/2}, \quad (4g)$$
N = 10 for rigid bottom at finite depth,
N = 100 for infinite subgrade,
PP = $\pi A^2 p$ (p = tire pressure), and
μ = Poisson factor.

Equations 2, 3, and 4–4g are used iteratively by pocket computer to determine the pavement modulus E^* for any given combination of subgrade modulus $E\phi$, pavement thickness HC, load PP, tire pressure p, and center deflection $D\phi$.

STRENGTHENING DESIGN OF PIURA AIRPORT

Destructive Testing Results

The test results of representative samples of subgrade, subbase, and base course materials are shown in Table 1. This investigation was carried out on six test pits, the locations along the pavement facilities of which are shown in Figure 7. The test results shown in Table 1 indicate that the pavement materials do not meet standard Federal Aviation Administration (FAA) specifications and are very sensitive to increased moisture. The increase in moisture analyzed in Table 1 was obtained by immersing the sample for 1 day. For example, in Pits 4 and 5 the base course material is classified as GC (clayey gravel or clayey sandy gravel) with plastic index PI = 9–19 and Passing Sieve 200 = 25–35. Standard specifications call for PI equal to or less than 6 and Passing Sieve 200 between 2 and 10. As is also shown in Table 1, the CBR of the GC material reduces from the 50–100 range to 2–9 after 1 day of immersion. Standard specifications call for minimum CBR = 80 after 4 days of immersion. Similar conclusions were obtained in other test pits.

Such routine destructive tests could not explain why the pavement of Piura Airport can accommodate medium-sized B-727, B-737, and F-28 jet aircraft. The only practical way to explain the past performance of the Piura pavement systems and to assign rational strength to the existing materials was to implement NDT techniques.

Analysis of NDT

The NDT survey and analysis determine the maximum deflection $D\phi$, offset deflection DR at an offset distance R, characteristic length $L\phi$, subgrade modulus $E\phi$, subgrade CBR, and pavement modulus E^*. Tables 2 and 3 present typical computerized results determined on Piura runway between Stations 1 + 375 and 1 + 975. The NDT was carried out each 25 m. The results in Tables 2 and 3 refer to offset deflection of R = 40 cm and R = 70 cm, respectively. These tables indicate that in both cases the distribution of CBR along the runway is similar. For both offset distances, the subgrade CBR is about 7 percent from Stations 1 + 375 to 1 + 575 and about 3 percent from Stations 1 + 575 to 1 + 975. This indicates that different deflections and

TABLE 1 INFLUENCE OF MOISTURE CHANGES ON MATERIAL STRENGTH (Piura Airport)

Pit No.	Type of Material	Classification	LL (%)	PI (%)	Passing Sieve 200 (%)	In Situ Moisture (%)	Laboratory CBR: In Situ Density (%) In Situ Moisture	Laboratory CBR: In Situ Density (%) 1 Day in Water
P-1	Subgrade	SC	30	12	44	3	73	2
P-2	Base	GM-GC	23	5	30	6	100	24
P-3	Subbase	SM	21	NP	20	2	100	9
P-3	Subgrade	SP-SM	–	NP	12	3	34	6
P-4	Base	GC	26	9	25	2	100	9
P-4	Subgrade	ML-CL	25	6	75	4	39	1
P-5	Base	GC	32	19	35	8	50	2
							100	3
P-6	Base	GP-GM	14	NP	10	1	82	8

NOTE: LL = liquid limit, PI = plastic index, CBR = California bearing ratio. SC = clayey sand or clayey gravelly sand; GM = silty gravel or silty sandy gravel; GC = clayey gravel or clayey sandy gravel; SM = silty sand or silty gravelly sand; SP = sand or gravelly sand, poorly graded; ML = silts, sandy silts, gravelly silts, or diatomaceous soils; CL = lean clays, sandy clays, or gravelly clays; and GP = gravel or sandy gravel, poorly graded.
SOURCE: Corps of Engineers.

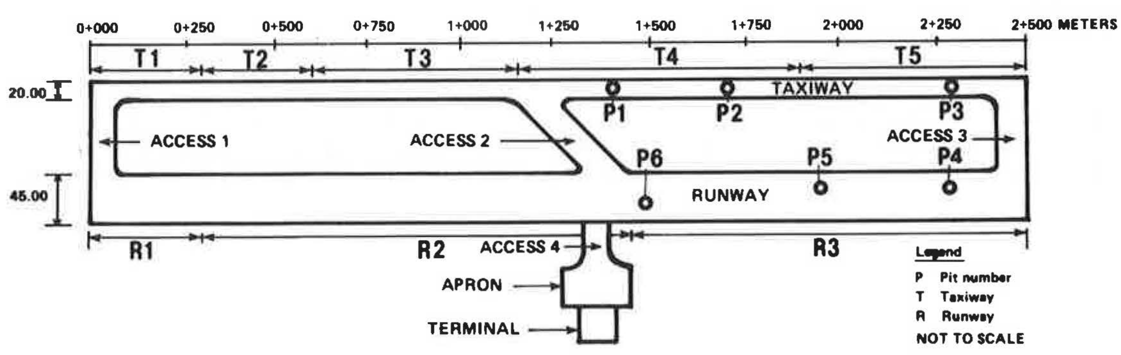

FIGURE 7 Schematic layout of Piura Airport.

TABLE 2 NDT TEST RESULTS OF PIURA RUNWAY (R = 40 cm)

Station	DØ .01 mm	DR .01 mm	LØ (cm)	EØ (kg/cm²)	CBR (%)	E* (kg/cm²)	E*/EØ	CBR Graphic
1+375	50.0	32.0	29.1	676.4	6.1	2526.5	3.7	******
1+400	58.0	30.0	20.0	747.2	6.8	1505.0	2.0	******
1+425	·58.0	38.0	30.4	563.8	5.1	2298.9	4.1	*****
1+450	36.0	26.0	36.9	773.7	7.0	4797.8	6.2	******
1+475	38.0	30.0	46.0	605.7	5.5	6329.0	10.4	******
1+500	46.0	,36.0	44.9	511.2	4.6	5041.9	9.9	*****
1+525	44.0	32.0	37.4	624.8	5.7	3996.5	6.4	**
1+550	66.0	56.0	58.3	282.2	2.6	5374.1	19.0	**
1+575	64.0	54.0	57.0	296.7	2.7	5360.2	18.1	**
1+600	76.0	54.0	35.6	377.5	3.4	2156.8	5.7	**
1+625	80.0	56.0	34.5	368.1	3.3	1959.2	5.3	**
1+650	58.0	54.0	96.7	200.6	1.8	14963.6	74.6	**
1+675	74.0	46.0	27.7	475.5	4.3	1613.6	3.4	** *
1+700	90.0	62.0	33.4	336.1	3.1	1690.4	5.0	**
1+725	82.0	50.0	26.8	440.0	4.0	1385.7	3.1	** *
1+750	104.0	68.0	30.3	315.4	2.9	1270.5	4.0	**
1+775	96.0	60.0	27.9	363.9	3.3	1252.7	3.4	**
1+800	114.0	68.0	25.9	325.3	3.0	961.0	3.0	**
1+825	130.0	58.0	17.1	379.6	3.5	560.7	1.5	**
1+850	184.0	128.0	34.1	161.8	1.5	845.1	5.2	*
1+875	124.0	82.0	30.9	260.1	2.4	1098.7	4.2	**
1+900	110.0	74.0	31.9	285.6	2.6	1290.2	4.5	**
1+925	104.0	68.0	30.3	315.4	2.9	1270.5	4.0	**
1+950	144.0	72.0	19.9	310.8	2.8	580.5	1.9	**
1+975	158.0	92.0	24.9	241.6	2.2	666.5	2.8	**

NOTE: $D\phi$ = maximum deflection under rear axle, DR = deflection at a distance R from $D\phi$ or rear axle, $L\phi$ = characteristic length (distance at which deflection is 50 percent of $D\phi$), $E\phi$ = subgrade modulus of elasticity, CBR = California bearing ratio, $E*$ = pavement modulus.

TABLE 3 NDT TEST RESULTS OF PIURA RUNWAY (R = 70 cm)

Station	DØ .01 mm	DR .01 mm	LØ (cm)	EØ kg/cm²	CBR (%)	E* kg/cm²	E*/EØ	CBR Graphic
1+375	50.0	14.0	22.7	816.8	7.4	1904.3	2.3	******
1+400	58.0	16.0	22.5	709.5	6.5	1619.6	2.3	******
1+425	58.0	12.0	18.6	808.9	7.4	1352.8	1.7	******
1+450	36.0	14.0	29.5	925.9	8.4	3560.4	3.8	******
1+475	38.0	18.0	36.0	748.2	6.8	4420.4	5.9	******
1+500	46.0	20.0	32.7	669.9	6.1	3189.3	4.8	******
1+525	44.0	18.0	30.9	731.8	6.7	3091.1	4.2	**
1+550	66.0	32.0	37.0	420.9	3.8	2651.4	6.3	**
1+575	64.0	40.0	52.1	322.0	2.9	4622.0	14.4	**
1+600	76.0	34.0	33.7	395.0	3.6	2025.3	5.1	** *
1+625	80.0	38.0	36.1	354.4	3.2	2094.0	5.9	**
1+650	58.0	34.0	47.3	387.0	3.5	4346.5	11.2	** *
1+675	74.0	22.0	23.7	534.6	4.9	1344.2	2.5	** **
1+700	90.0	34.0	28.8	379.3	3.4	1379.7	3.6	**
1+725	82.0	26.0	24.9	465.2	4.2	1283.5	2.8	** *
1+750	104.0	32.0	24.4	373.2	3.4	993.1	2.7	**
1+775	96.0	32.0	35.9	385.8	3.5	1139.6	3.0	** *
1+800	114.0	38.0	25.9	324.9	3.0	959.7	3.0	**
1+825	130.0	46.0	27.2	274.4	2.5	890.9	3.2	**
1+850	184.0	48.0	21.6	230.0	2.1	491.3	2.1	**
1+875	124.0	40.0	25.3	304.6	2.8	855.1	2.8	**
1+900	110.0	38.0	26.7	329.3	3.0	1037.1	3.1	**
1+925	104.0	32.0	24.4	373.2	3.4	993.1	2.7	**
1+950	144.0	40.0	22.6	284.8	2.6	657.0	2.3	**
1+975	158.0	42.0	21.9	265.[illegible]	2.4	579.8	2.2	**

NOTE: $D\phi$ = maximum deflection under rear axle, DR = deflection at a distance R from $D\phi$ or rear axle, $L\phi$ = characteristic length (distance at which deflection is 50 percent of $D\phi$), $E\phi$ = subgrade modulus of elasticity, CBR = California bearing ratio, $E*$ = pavement modulus.

offset distances can be used and still result in the same CBR or subgrade modulus. This can also be considered another verification of the NDT methodology used in this study. This verification is shown in more detail in Figure 8, which shows the relationship between the CBR calculated for both $R = 40$ cm and $R = 70$ cm. Up to CBR = 8, both calculated CBR values are practically the same. For higher CBR values, which are associated with strong subgrade and smaller deflections, the deflection measurement is more sensitive and therefore the correlation is, relatively, not as good. In conclusion, the subgrade CBR values calculated from the NDT vary between the upper and lower CBR values shown in Table 1 and seem to accurately represent the performance of the local subgrade materials of Piura Airport.

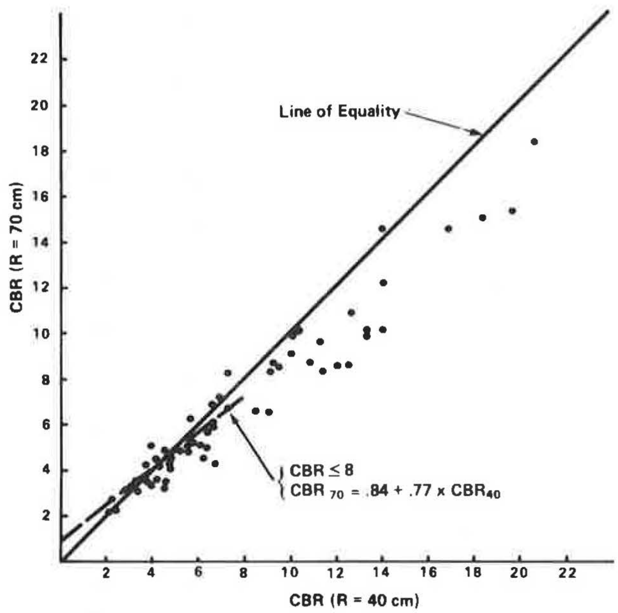

FIGURE 8 Distribution of subgrade CBR for $R = 40$ and $R = 30$ cm.

Pavement Evaluation and Strengthening Design

The total pavement thickness (*HC*, see Figure 6) is input to calculate the pavement modulus E^* (Equations 3 to 4g). Increasing the *HC* will reduce the value of E^* for a given subgrade modulus and deflection basin. The granular pavement modulus depends also on the subgrade strength *(15)*. In other words, $E_2/E\phi$ or $E^*/E\phi$ (see Figure 6) depends on $E\phi$. For a very strong subgrade E_2/E_1 tends to reach the Boussinesq uniform media where $E_2/E_1 = 1$. Experience with flexible pavement indicates that adequate performance is achieved when $E_2/E\phi$ varies between 2 and 5 *(2–4, 9, 15)*. This was also confirmed in the Piura study. No subgrade shear failure was observed when $E^*/E\phi$ was over 2 and subgrade CBR was over 6. The lower the subgrade CBR the higher the moduli ratio should be to prevent shearing failure and to achieve adequate pavement performance. Figure 9 presents a typical relationship between $E^*/E\phi$, *HC*, and subgrade CBR. This relationship was developed for the existing Piura Airport pavement sections that show adequate pavement performance. The main conclusion of Figure 9 is that for any given pavement thickness *HC*, an increase in the subgrade CBR reduces the required moduli ratio

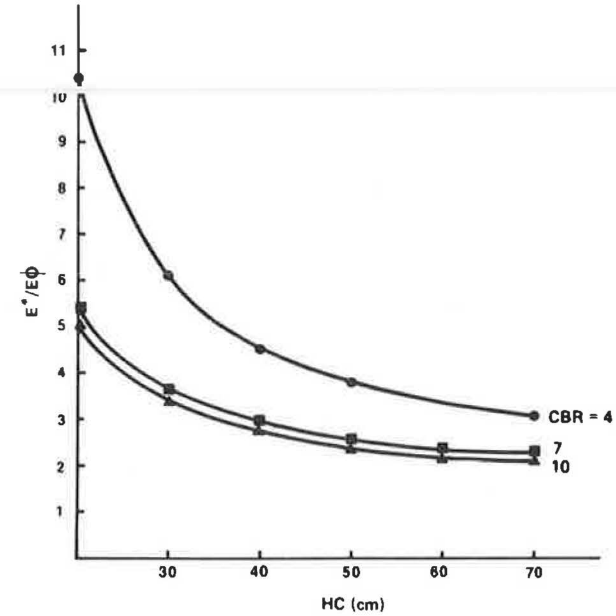

FIGURE 9 Typical relationship among $E^*/E\phi$, HC, and the subgrade CBR.

$E^*/E\phi$. Another interpretation of Figure 9 is that if the subgrade CBR and the moduli ratio $E^*/E\phi$ are known, the equivalent pavement thickness H_{EC} can be calculated. H_{EC} can be larger and smaller than the measured pavement thickness if $E^*/E\phi$ is greater than 5 and less than 2, respectively. The mechanism to determine the effective existing pavement thickness—H_{EC} in the Piura rehabilitation program—was carried out by means of the factor α defined in Equation 5:

$$\alpha = (\mathrm{CBR}) \times (E^*)/(E\phi) \quad (5)$$

The higher the α factor, the higher the pavement rigidity or the better the existing pavement. Based on the conclusions shown in Figure 9 and the modulus ratio criteria of $E_2/E\phi = 2 - 5$ *(2–4, 9, 15)*, the following relationship between α and the existing pavement thickness *HC* was developed:

1. When $\alpha \leq 10$ *HC* is reduced 10–20 percent, or $H_{EQ} =$ (0.8–0.9) *HC*;
2. When $10 > \alpha > 15$, *HC* is not modified, or $H_{EQ} = HC$; and
3. When $\alpha \geq 15$, *HC* is increased 10–20 percent, or $H_{EQ} =$ (1.1–1.2) *HC*.

HC denotes the existing pavement thickness, which was approximately 40 cm. H_{EQ} denotes the equivalent pavement thickness used to upgrade the existing pavement to carry 600 annual operations of B-727-100, with gross aircraft weight of 160,000 lb. The design parameters obtained from the NDT survey for the pavement strengthening are the subgrade CBR and α. The total thickness of a flexible pavement H_N needed to carry this traffic loading on a given subgrade CBR was determined according to the FAA standard guidelines *(16)*. The difference between the needed H_N and the existing equivalent thickness H_{EQ} is the required pavement strengthening. Table 4 summarizes the strengthening design of Piura Airport. The airport was divided into different sections with similar design parameters. For example, Section *R2* is located on the runway between Stations 0 km + 300 m and 1 km + 450 m. The subgrade CBR is equal to 7 and $\alpha = 20$. $H_N = 70$ cm *(16)*. The existing equivalent pavement thickness was increased from $HC = 40$ to 50 cm, because α is equal to 20. The $70 - 50 = 20$ cm of difference in pavement thickness is equivalent to 10 cm of asphalt concrete and 10 cm of granular base, which is equivalent to 5 cm of black base *(16)*. The Piura Airport pavement strengthening was completed in November 1986.

EVALUATION AND UPGRADING OF TALARA AIRPORT

This airport is located about 120 km north of Piura near the town of Talara. The altitude of the airport is about 100 m above sea level; the runway length is 2460 m; and the direction is 17/35. Takeoff and landing are done from north to south because of the strong southern wind. The climate conditions in Talara are similar to those in Piura with the water table 5–8 m beneath the subgrade surface. At Talara Airport, the use of marginal local materials has resulted in acceptable pavement performance. This airport is currently used for DC-8, B-727, and F-28 jet aircraft. The thickness of the upper layer of AC is 6 to 7 cm. The total airport pavement thickness varies between 45 and 60 cm, with an average total thickness of approximately $HC = 50$ cm. The pavement provided acceptable service during the last 10 yr. The base course is silty sand and gravel and its thickness is 15–30 cm. This base course has the following engineering properties (see Table 1 for classification description):

Classification: GC, GP-GM, and SP-SM;
Natural moisture: 4.5–5.5 percent;
Passing sieve 200: 19–25 percent (according to the FAA, it should be 2–10 percent); and
Plastic index: 7–14 (should be less than 7).

The laboratory CBR of the base course is very sensitive to moisture. An increase in humidity from 4.8 to 6.5 reduces the laboratory CBR from 80 to 10. The existing subbase material is sand clay or sandy silt classified as SC, SP-SM. The natural moisture content varies between 5 and 7 percent, passing Sieve 200 is 30 to 40 percent (should be less than 20 percent), and the plastic index is 7 to 15 percent (should be less than 8 percent). The local subgrade material is classified as CL, SP (see Table 1). Moisture content in the subgrade varies between 2 and 9 percent and the in situ CBR varies mainly between 5 and 8.

The preceding engineering information indicates that both the subbase and base-course materials do not meet standard FAA specifications and it is known that in tropical or even subtropical conditions this pavement is simply not adequate. Nevertheless, because of the arid conditions of Talara, good performance has been achieved. The NDT study carried out from December 1984 to January 1985 was analyzed and interpreted to better understand the performance of these marginal materials. The NDT survey was also used to determine the true strength parameters of the pavement materials. The conclusions of the NDT survey are presented in Figures 10 and 11. Figure 10 presents the CBR distribution and Figure 11 presents the distribution of α values defined in Equation 5.

Good correlation was obtained between the calculated CBR from the NDT survey and the in situ CBR determined on Pits 2 and 5 (see Figure 10). The in situ subgrade CBR was 6 and 11 in these two pits. Indicated in Figures 10 and 11 are the following three categories of subgrade CBR and three categories of α: CBR ≤ 5; $5 <$ CBR ≤ 8, CBR > 8, and $\alpha \leq 10$, $10 < \alpha$

TABLE 4 STRENGTHENING PAVEMENT DESIGN: PIURA AIRPORT

Pavement Section[a]	Subgrade CBR (%)	α	H_N—Total Pavement Thickness Needed (cm)[b]	H_{EC}—Existing Pavement Thickness (cm)	Thickness of Strengthening Layers (cm)				Comments
					AC	Base	Subbase	Concrete Slab	
Runway Section									
(R1) 0 + 000 to 0 + 300	–	–	–	–	10	–	–		Concrete slab
(R2) 0 + 300 to 1 + 450	7	20	70	50	10	5[c]	–		
(R3) 1 + 450 to 2 + 500	4	8	90	30	10	20	30		
Taxiway									
(T1) 0 + 000 to 0 + 300	–	–	–	–	10	–	–		Concrete slab
(T2) 0 + 300 to 0 + 600	4	10	90	35	10	20	25		
(T3) 0 + 600 to 1 + 150	6	14	70	40	10	20	–		
(T4) 1 + 150 to 1 + 900	4	8	90	30	10	20	30		
(T5) 1 + 900 to 2 + 500	7	15	65	45	10	5[c]	–		
Access 1	9	20	60	50	10	–	–		
Access 2	7	15	65	45	10	5[c]	–		
Access 3	7	New	65	–	10	20	30		New pavement
Access 4	6	12	70	40	10	10[c]	–		
Apron	7	14	65	40	10	8[c]	–		

[a]See Figure 9 for location and layout.
[b]See *FAA Manual (16)*.
[c]Asphalt base.

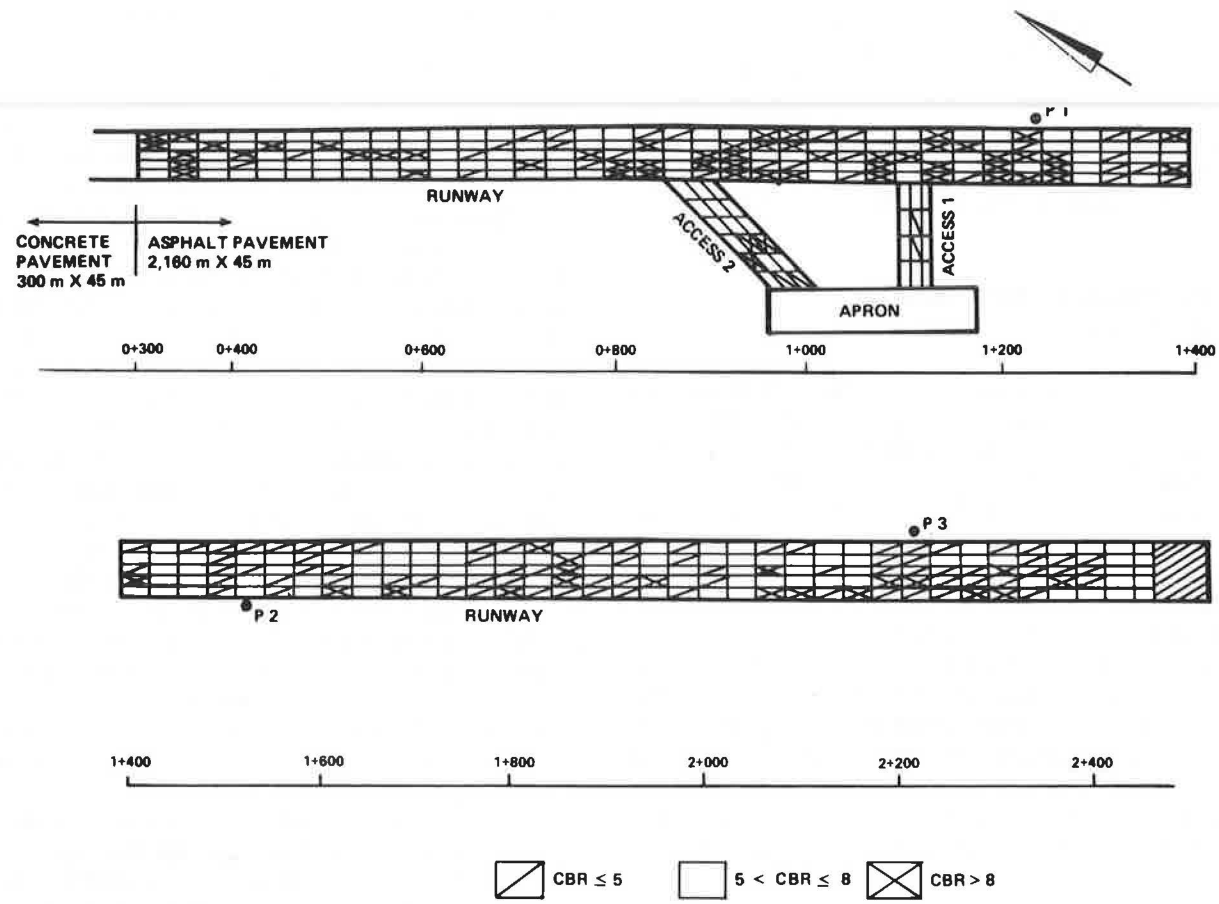

FIGURE 10 Distribution of the subgrade CBR in Talara Airport.

< 15, α ≥ 15. The average representative design parameters of the Talara Airport are CBR = 6–8 and α = 10–15. The average equivalent total pavement thickness design is H_{EC} = 50 cm.

Such an airport pavement can carry about 500 annual departures during 10 yr of a dual wheel gear aircraft with total gross weight of 100,000–120,000 lb *(16)*. This seems to be the weight of partially loaded B-727s, B-737s, and F-28s, mostly used for short distances of about 1100 km, which is the distance from Lima to Talara. In 1986, the Peruvian Transport Authority and the World Bank decided to implement the NDT survey to upgrade the airport pavement so that it could be used for B-727-100s with a gross weight of 160,000 lb and 600 annual departures. The strengthening design was based on (a) the previously mentioned traffic loading, (b) the distribution of CBR and α given in Figures 10 and 11, and (c) the FAA thickness design guidelines *(16)*.

The methodology used at Talara is identical to the strengthening design procedure used at Piura and is presented in Table 5. For example, in Station 0 km + 300 m to 1 km + 200 m, the subgrade CBR is 8 and α is 10. The total flexible pavement thickness needed is H_N = 60 cm *(16)*. As α = 10, the existing equivalent pavement thickness is reduced from 50 to 45 cm.

TABLE 5 STRENGTHENING PAVEMENT DESIGN: TALARA AIRPORT (600 annual operations—B-727)

Pavement Section	Subgrade CBR (%)	α	H_N—Total Pavement Thickness Needed (cm)[a]	H_{EC}—Existing Pavement Thickness (cm)	AC Overlay (cm)
Runway Section					
0 to 0 + 300	*K* = 150–250 PCI	Rigid	33	27	9
0 + 300 to 1 + 200	8	10	60	45	9
1 + 200 to 1 + 800	7	12	65	50	9
1 + 800 to 2 + 460	6	15	71	55	9
Accesses					
Access 1	7	15	65	55	6
Access 2	7	15	65	55	6

[a]See *FAA Manual (16)*.

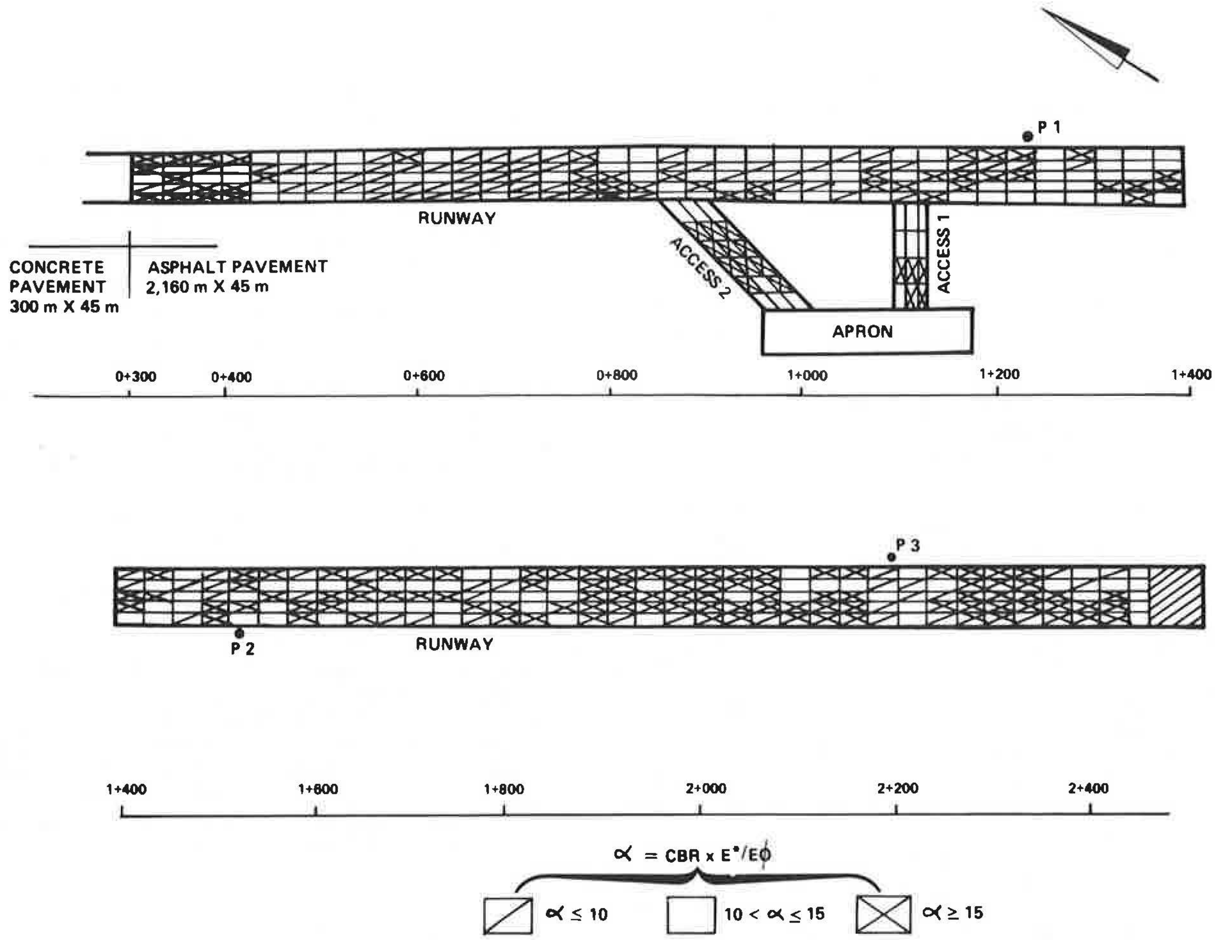

FIGURE 11 Distribution of α in Talara Airport.

The difference of pavement thicknesses is ($H_N = 60$) – ($H_{EC} = 45$) = 15 cm, which is equivalent in the semiarid region of Peru to 9 cm of asphalt overlay.

In a similar way, 9 cm of AC was designated to strengthen other sections of the runway and 6 cm of AC was needed to strengthen Accesses 1 and 2. The overlay thickness design of 9 cm of AC was also used to strengthen the rigid northern runway section *(16)* (see Table 5). This thickness might not be sufficient to prevent reflection of cracks. Nevertheless, it was decided to construct this AC overlay with two layers: the bottom AC layer of 6 cm with coarse gradation and a higher percentage of air voids of 6–8 percent to reduce refractions of cracks, and the upper layer of 3 cm with 3–5 percent air voids with smaller particles to improve riding quality. Monitoring of the new pavement performance is scheduled after construction is completed.

SUMMARY AND CONCLUSIONS

1. The Piura and Talara Airports have 2500-m-long runways and are located in the northern Pacific region of Peru, 1000–1100 km north of Lima. The water table is 2–8 m under the subgrade surface and the annual rainfall is less than 150 mm. Currently, B-727s, B-737s, and F-28s are using both airports and DC-8s are also used at Talara.

2. The existing pavement materials do not meet standard FAA specifications related to plasticity, gradation, and CBR-moisture relationships. Nevertheless, experience of about 15 yr generally indicates adequate pavement performance.

3. Local pavement failure occurred in 1982 in the southern edge of Piura Airport because of a combination of pavement overstress and rare flash flooding with a return period of over 100 yr.

4. It is concluded that the most practical way to interpret the actual performance of marginal pavement materials in the Peruvian arid zone is to use NDT to determine the elastic parameters of the existing subgrade and pavement materials.

5. The methodology used to evaluate and upgrade the Piura and Talara Airports is based on the Hogg model of a thin plate on an elastic foundation. The subgrade modulus can be determined without prior knowledge of the thickness or other characteristics of the pavement layers. Once the subgrade modulus is known, the pavement modulus can be calculated for any given load and center deflection.

6. The interpretation of the NDT survey indicates that the existing marginal pavement materials in the arid zone that do not meet standard specifications can generate sufficient bearing capacity to support traffic loads of B-727s, B-737s, F-28s, and DC-8s.

7. The subgrade modulus, CBR, and pavement modulus were determined each 25 m by an NDT survey carried out with a Benkelman beam. The pavement parameters were used to upgrade the existing airports so that they could carry [illegible] annual operations of B-727s with total gross weights of 160,000 lb.

REFERENCES

1. J. Greenstein. Use of NDT and Pocket Computers in Pavement Evaluation. *ASCE Journal of Transportation Engineering*, Vol. III, No. 3, May 1985.
2. G. Wiseman and J. Greenstein. Comparison of Methods of Determining Pavement Parameters from Deflection Bowl Measurements. *Proc., Seventh Asian Regional Conference on Soil Mechanics and Foundation Engineering*, Haifa, Israel, Aug. 1983, pp. 158–165.
3. L. Berger and J. Greenstein. Use of Pocket Computers for Rehabilitation of Rural Roads in Dominica. In *Transportation Research Record 997*, TRB, National Research Council, Washington, D.C., 1984, pp. 90–98.
4. G. Wiseman, J. Greenstein, and J. Uzan. Application of Simplified Layered Systems to NDT Pavement Evaluation. In *Transportation Research Record 1022*, TRB, National Research Council, Washington, D.C., 1985, pp. 29–36.
5. A. H. A. Hogg. Equilibrium of a Thin Plate, Symmetrically Loaded Resting on an Elastic Foundation of Infinite Depth. *Philosophic Magazine*, Vol. 25, No. 168, 1938.
6. A. H. A. Hogg. Equilibrium of a Thin Slab on an Elastic Foundation of Finite Depth. *Philosophic Magazine*, Vol. 35, No. 243, 1944.
7. W. T. Hoyinck, R. Van Den Ban, and W. Gerritsen. Lacroix Overlay Design by Three-Layer Analysis. *Proc., Fifth International Conference on the Structural Design of Asphalt Pavements*, Delft, The Netherlands, Vol. 1, Aug. 1982, pp. 410–420.
8. G. Wiseman and L. Berger. Evaluation of Airfield Pavements and Subgrades Based on Deflection Basin Measurements (NDT). *Report to TRB Task Force A2T56—Nondestructive Evaluation of Airfield Pavements*, TRB, National Research Council, Washington, D.C., 1981.
9. G. Wiseman, J. Uzan, M. S. Hoffman, I. Ishai, and M. Livneh. Simple Elastic Models for Pavement Evaluation Using Measured Surface Deflection Bowls. *Proc., Fourth International Conference on Structural Design of Asphalt Pavements*, Ann Arbor, Mich., Vol. 2, 1977, pp. 416–426.
10. J. L. Green and J. W. Hall, Jr. *Nondestructive Vibratory Testing of Airport Pavements*. Experimental Test Results and Development of Evaluation Methodology and Procedure. Report FAA-RD-205, Publication WES TR S-75-14. Vol. 1, Federal Aviation Administration, U.S. Department of Transportation, Sept. 1975.
11. W. Heukelom and A. F. G. Klomp. Dynamic Testing as a Means of Controlling Pavements During and After Construction. *Proc., Fifth International Conference on the Structural Design of Asphalt Pavements*, Delft, The Netherlands, 1982.
12. G. Wiseman. The Interpretation of Surface Deflection Measurements Using the Model of an Infinite Plate on an Elastic Foundation. *Symposium on Nondestructive Test and Evaluation of Airport Pavement*, U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, Miss., Nov. 1975.
13. P. Ullidtz. Overlay and Stage-by-Stage Design. *Proc., Fourth International Conference on the Structural Design of Asphalt Pavements*, University of Michigan, Ann Arbor, 1977.
14. P. Ullidtz and K. R. Peattie. Pavement Analysis by Programmable Calculators. *Journal of the Transportation Engineering Division*, ASCE, Vol. 106, No. TE5, Sept. 1980, pp. 581–597.
15. A. I. M. Claessen, J. M. Edwards, P. Sommer, and P. Uge. Asphalt Pavement Design: The Shell Method. *Proc., Fourth International Conference on the Structural Design of Asphalt Pavements*, Vol. I, University of Michigan, Ann Arbor, 1977, pp. 39–44.
16. *FAA Manual*. AC-150/5320-6C, Federal Aviation Administration, U.S. Department of Transportation, 1978.

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