Evaluation of Roller-Compacted Concrete Pavements Using Nondestructive Load Testing

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Nondestructive load testing was carried out to study the behavior of roller-compacted concrete (RCC) pavements located at Moran and Conley Terminals, Boston, Massachusetts. Tests were conducted in late October 1990 with a Dynatest 8081 heavy weight deflectometer. Loads were applied at various locations on each of the 11 selected test sections. Measured deflections were first used to estimate the in situ pavement parameters. The estimated values were then used, in conjunction with a concrete pavement analysis program, to evaluate the structural performances of the pavement sections. Also, load transfer efficiency across joints and cracks was evaluated, and laboratory tests were performed on cores taken from the pavements. From the study it was found that RCC generally possessed engineering properties similar to those of conventional concrete. However, the RCC modulus of elasticity appeared to be lower than that of portland cement concrete. Large variations in load transfer efficiency were observed. The structural performance of the RCC pavement test sections in the study appeared to be adequate.

Since the first large-scale construction of roller-compacted concrete (RCC) pavement in British Columbia, Canada, in 1976 the use of RCC pavements has gained great popularity in recent years. The primary uses of RCC as paving material have been for off-highway facilities and for heavy-duty secondary roads. It is generally agreed among pavement engineers that it is less expensive to construct RCC pavements than asphalt concrete (AC) and portland cement concrete (PCC) pavements, and savings of 30 percent can usually be expected. The maintenance costs for RCC pavements are also less than those for AC pavements. Another important advantage over AC pavements offered by RCC pavements is their resistance to chemical attack from hydraulic fluid, fuel, and other hydrocarbons.

Because RCC was a relatively new material and no specific design methods were available, most of the RCC pavements were designed following the guidelines used for designing conventional PCC pavements. This was based on the assumption that RCC possesses engineering properties similar to those of PCC. As part of a research effort sponsored by the Portland Cement Association (PCA) to advance the technology for RCC pavements, nondestructive load testing was conducted to evaluate the RCC pavements at Moran and Conley Terminals, Boston, Massachusetts. The objective of this research work was to study RCC pavement behavior under loading, and thus to assess the suitability of using conventional pavement thickness design procedures for RCC pavements.

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BACKGROUND

The use of RCC pavement was pioneered in North America by the U.S. Army Corps of Engineers. In 1975 a test section of 3.7 × 32.0 m (12 × 105 ft) was placed by the Waterways Experiment Station as part of a street in Mississippi (*1*). However, the first large-scale construction of RCC pavement, a 16,000-m^2 (4-acre) log-sorting yard, took place in British Columbia, Canada, in 1976 (*2*). The pavement slabs, with a thickness of 355 mm (14 in.), were placed in two lifts. Slab thickness design was based essentially on engineering judgment and experience obtained from cement-stabilized base. No joints other than construction joints were provided. Slabs were allowed to crack naturally, and the spacing was generally in the range of 12 to 18 m (40 to 60 ft).

Following the success of the first application, several other log-sorting yards in British Columbia were constructed with RCC. All of the pavements had the same thickness, 355 mm (14 in.), as the first pavement, and the same design concept used for the first pavement was used. Usually, the remainder of the structural system consisted of a 150-mm (6-in.) granular subbase placed over a consolidated subgrade. The design method for RCC pavements currently used in Canada is the PCA's airport thickness design procedure.

In the United States the first production project of RCC pavement, a test section 71.4 m (234 ft) long by 6.1 m (20 ft) wide, was constructed at Fort Stewart, Georgia, in July 1983 (*3,4*). The pavement had a slab thickness ranging from 230 to 330 mm (9 to 13 in.) and currently serves as an access from a tracked-vehicle parking area to a series of tank trials. In July 1984 a 15 000-m^2 (18,000-yd^2) RCC parking area was constructed at Fort Hood, Texas. This 255-mm (10-in.)-thick pavement was designed to carry 54 000-kg (120,000-lb) tracked vehicles as normal traffic.

Because of their early success, RCC pavements have been used on several large-scale projects since 1985. These projects include the following:

- An aircraft parking apron at Portland International Airport, Portland, Oregon, 1985 (*5*).
- An intermodal yard (Rennick Yard) at Denver, Colorado, 1986 (*6*).
- An army base at Fort Drum, New York, 1988 and 1989 (*7*).
- Various projects at Tasmania, Australia, 1986 and 1987 (*8*).
- High-speed test sections near Melbourne, Australia, 1988 and 1989 (*9*).

The use of conventional rigid pavement design procedures for RCC pavements was based on the assumption that RCC possesses engineering properties similar to those of PCC. This assumption was later justified by strength testing conducted on cores and laboratory specimens obtained from various RCC pavements. However, RCC is likely to show lower density, and hence lower strength, in the areas near construction joints, where compaction is often less effective because of a lack of edge support.

In a laboratory study to evaluate RCC material properties, Tayabji and Okamoto (*10*) also concluded that the engineering behavior of RCC was similar to that of conventional normal-weight concrete. Based on that study a pavement thickness design procedure for RCC was developed (*11*). This procedure essentially followed the same concept used in conventional rigid pavement design, with the substitution of a different fatigue curve. The specific RCC fatigue relationship, which was similar to the PCC fatigue relationship, was developed from RCC beams sawed from full-scale RCC test panels.

FIELD AND LABORATORY TESTING

Description of Test Sections

Both Moran and Conley Terminals are located at Boston Harbor, Massachusetts, serving as container storage and transshipping yards. RCC pavements were designed to support the weights of the containers as well as the equipment used to transport them. The primary transporting device used at Moran is a special-purpose front-end loader, the Marathon LeTourneau Letro-Porter (nicknamed Hurdy-Gurdy at the site). This is an extremely heavy machine with a maximum rated single wheel load of about 400 kN (90,000 lb). Several Letro-Porters were in use at Moran at the time of testing. Subsequently, one was transferred to Conley, which up to then had been solely a tractor and trailer operation.

It was reported that the Moran RCC pavements consisted of an RCC layer of 380 mm (15 in.), a 230-mm (9-in.) gravel subbase, and a compacted subgrade. RCC slabs were constructed in three lifts of 140, 140, and 100 mm (5.5, 5.5, and 4 in.) from bottom to top. No transverse contraction joints except construction joints were provided during construction. Instead, the slabs were allowed to crack naturally. It was somewhat surprising to note that very long crack spacing (greater than 30.5 m or 100 ft) was not uncommon at the Moran RCC pavement site. Both the construction joints and cracks showed widely different performances. Some have remained in relatively good condition, whereas others have badly deteriorated. Slab width varied, ranging from 4.3 to 8.2 m (14 to 27 ft).

Four RCC pavement sections were selected for testing. They were designated Slabs M1 through M4. Slab dimensions were about 30.5 × 4.9 m (100 × 16 ft), 30.5 × 8.2 m (100 × 27 ft), 39.0 × 8.2 m (128 × 27 ft), and 30.5 × 4.6 (100 × 15 ft) for Slabs M1, M2, M3, and M4, respectively. Although Slabs M1, M2, and M3 were enclosed by construction joints and cracks, two sides of Slab M4 were adjacent to an area of asphalt pavement, thus approximating the free-edge condition. Deflections for determination of load transfer efficiency (LTE) were also measured at locations throughout the site. LTE was defined as the deflection measured at the unloaded side divided by that measured at the loaded side, expressed as a percentage.

The design thickness of the Conley RCC slab was 455 mm (18 in.), constructed in three lifts of 165, 165, and 125 mm (6.5, 6.5, and 5.0 in.) from bottom to top. A 200-mm (8-in.)-thick dense graded, crushed stone base was placed under the RCC slab. The subgrade material was mainly composed of sand, silt, and cobbles.

Seven RCC pavement sections, designated Slabs C1 through C7, were selected for testing. Dimensions for Slabs C1 to C7 were 19.5 × 5.1, 29.0 × 3.7, 36.3 × 5.1, 15.3 × 4.8, 15.3 × 5.1, 44.5 × 5.6, and 44.5 × 5.8 m (64 × 16.7, 95 × 12, 119 × 16.7, 50 × 15.8, 50 × 16.7, 146 × 18.3 and 146 × 19 ft), respectively. Additional locations were also chosen for load transfer deflection measurements.

Dynamic Load Testing

Tests were performed in late October 1990. A Dynatest 8081 heavy weight deflectometer (HWD) was used to provide the dynamic loads in the tests. The HWD is a device similar to a falling weight deflectometer, which delivers an impulse load to the pavement surface through a loading plate. However, the HWD has a much higher load capacity [greater than 265 kN (60 kips)], which is needed to generate realistically measurable pavement responses in such thick sections. The diameter of the loading plate is 450 mm (17.7 in.). Dynamic loads were applied at positions along both longitudinal and transverse joints and along transverse lines 4.9 m (16 ft) from the transverse joints. Loads were also placed at adjacent slabs along the transverse joints. This was done to evaluate if the LTE would stay the same with loads applied at different sides of the joints.

Deflections were measured by velocity transducers placed at the center of the loading area and outward to 1.5 m (5 ft) at 0.3-m (1-ft) intervals. A sensor was also placed at the opposite side of the load, 0.3 m (1 ft) from the loading center. Four different load levels, approximately 90, 155, 200, and 265 kN (20, 35, 45, and 60 kips), were used for each load position. Two drops were performed for each load level. The typical loading and sensor layouts are displayed in Figure 1. Each load position was identified by a station number, such as S1.1, S3.1, and S12.2. Additional load transfer measurements were conducted on different types of joints on slabs located through the Moran and Conley RCC sites.

The weather was cloudy and cool and the RCC pavement surface temperature varied between 9°C and 11°C (48°F and 52°F) during the entire testing process.

Laboratory Testing

Three RCC cores were taken from each slab tested at Moran Terminal, whereas two cores were taken from each of the RCC test slabs tested at Conley Terminal. The diameter of the cores was 95 mm (3.75 in.). After initial examination of bonding between lifts and measurements of specimen lengths, all of the cores were cut into either two or three pieces. Laboratory tests were then conducted on all prepared cores to determine the unit weight, compressive strength, split tensile strength, and RCC modulus of elasticity. The direct shear test was also performed on a few specimens from the Moran RCC pavement. All of the tests conducted followed the standard procedures of ASTM.

ANALYSIS OF TEST RESULTS

Estimation of K and E_c from HWD Deflection Measurements

By using the backcalculation procedure ECOPP developed at Construction Technology Laboratories, Inc. (*12*), RCC pavement pa-

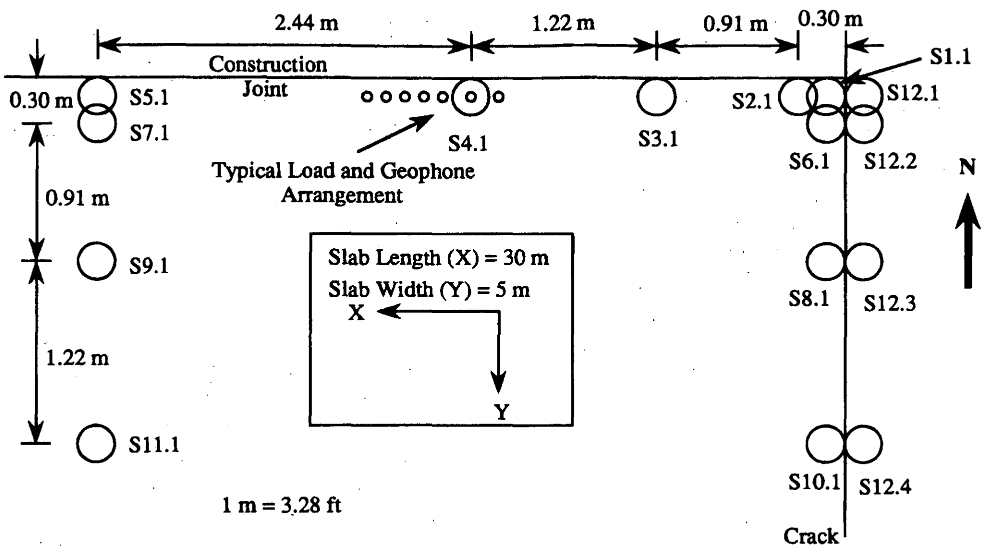

FIGURE 1 Typical HWD testing plan.

rameters were estimated on the basis of the deflections measured at the center load positions. In this procedure regression equations were developed from theoretical deflection data to estimate the RCC elastic modulus (E_c) and the modulus of subgrade reaction (K).

HWD deflections should be used with caution when attempting to backcalculate pavement material properties. It is a well-known fact that the temperature differential in a concrete slab will cause it to curl. When the top of the slab is warmer than the bottom, it will curl up at the slab interior and lose partial contact with the subbase or subgrade. Since deflections measured at slab interiors are usually used for the backcalculation process, the results might be misleading if deflections were collected under this situation.

To verify a complete support from the layer underneath, four load levels were used for each load position on the slabs. Slab M4 was excluded from the analysis since it was not subjected to a center load condition. An examination of the load-deflection characteristic for the center load position (S11.1) was made for each of the test slabs. It was observed that although a linear load-deflection relationship existed for Moran test slabs, Conley test slabs exhibited nonlinear behavior under center load conditions. The measured center load deflections are listed in Table 1.

Although showing some nonlinearity, attempts were still made to estimate the Conley RCC pavement parameters from the HWD deflections, and large variations in the estimated values were observed, especially for the lower load levels. However, for the two higher load levels, the estimated values showed some consistency. It was believed that the larger loads might have restored the pavement support condition, and the deflections under them were used in the analysis.

Actual average RCC slab thicknesses measured from cores were 400, 370, and 375 mm (15.8, 14.5, and 14.8 in.) for Slabs M1, M2, and M3, respectively, and they were 440, 510, 505, 470, 470, 480, and 470 mm (17.3, 20.0, 19.8, 18.6, 18.5, 18.9, and 18.6 in.) for Slabs C1 through C7, respectively. Table 2 lists the estimated elastic moduli and moduli of subgrade reaction for the RCC slabs at different loads. In general, the estimated values at different load levels were close to one another except those obtained at the 90-kN (20-kip) load level on Slab M1, which were excluded from the analysis. The average estimated RCC elastic moduli compared favorably with laboratory-measured values determined from cores except for the values for Slab C3, which were also excluded from further analyses.

By using predicted RCC moduli and moduli of subgrade reaction, the deflection basins for the four different loads were computed and compared with measured values. It was observed that the computed and measured deflection basins matched well at distances 3 ft or more from the loads. They did not match well close to the loads. Computed deflections were always greater than the measured ones. The discrepancy might be attributed to the stress-dependent behavior of subgrade material. It was hypothesized that the subgrade was stiffer when it was closer to the load center than away from it.

Following the concept of resilient modulus of soil and using a trial and error process, as described previously (*12*), variable moduli of the subgrade reactions for the RCC pavements were determined. The adjusted moduli of the subgrade reactions are displayed in Table 3. A typical comparison of the deflections computed by the concrete pavement analysis program JSLAB with adjusted K values and the measured deflections is shown in Figure 2. The deflection basins matched well with each other (Figure 2). Thus, the estimated values were considered satisfactory in representing in situ pavement material properties.

Analysis of RCC Pavement Response Data

Similar to the behavior of conventional concrete pavements, for all of the test slabs corner loads were observed to produce the highest maximum deflections, whereas the smallest maximum deflections occurred at interior loading conditions. In general, the deflections measured at the joint center or the slab edge were between these two cases. At the highest load level of 265 kN (60 kips), the interior maximum deflections were 58, 42, and 41 percent of the corner maximum deflections for Slabs M1, M2, and M3, respectively. The interior deflection measurement location in Slab M4 was only 0.6 m (2 ft) inward from the edge and, therefore, was not considered representative for a true interior loading condition. For Conley test sec-

TABLE 1 Measured HWD Deflections Under Center Load Condition

Slab No.	Load (kN)	Deflection (microns) Distance from Loading Center (m) 0.0	0.3	0.6	0.9	1.2	1.5	-0.3
M1	87	111	99	89	81	72	62	96
	158	180	169	150	140	125	109	164
	200	237	217	195	180	159	139	212
	268	298	274	246	226	201	175	267
M2	91	96	86	77	70	62	54	86
	164	174	154	138	127	114	100	151
	203	216	193	174	161	143	126	191
	282	279	243	218	199	177	155	241
M3	89	124	114	103	94	83	73	112
	160	219	198	181	170	150	133	195
	199	277	249	228	213	189	169	246
	267	351	312	283	258	232	207	302
C1	97	70	62	58	53	51	46	62
	176	125	108	100	95	87	78	107
	220	149	136	125	117	108	97	134
	292	180	167	150	140	131	116	164
C2	87	78	70	64	62	57	51	70
	157	136	122	112	104	98	87	124
	201	163	152	139	127	120	107	153
	265	196	183	170	157	142	128	184
C3	91	73	72	68	64	60	54	73
	166	146	127	115	109	101	90	125
	207	173	157	145	136	124	112	155
	281	211	191	176	165	149	131	190
C4	87	82	77	71	66	64	58	76
	157	144	132	123	118	108	100	133
	203	173	164	147	143	131	121	163
	268	209	196	178	170	156	142	196
C5	89	89	79	73	72	65	59	79
	161	158	139	128	121	111	103	136
	199	185	173	158	150	137	124	170
	275	219	207	186	172	160	146	204
C6	88	75	76	69	65	59	52	75
	157	147	132	121	116	104	93	130
	200	173	165	150	141	130	115	160
	264	217	197	181	169	153	137	191
C7	88	65	66	60	56	53	47	66
	162	147	117	107	100	92	81	116
	202	163	144	137	127	111	100	143
	275	198	178	165	152	134	117	174

1 kN = 225 lbf, 1 micron = $3.937*10^{-5}$ in., 1m = 3.28 ft

tions, at the highest load level of 265 kN (60 kips), the interior maximum deflections were 14, 17, 36, 34, 31, and 23 percent of the corner maximum deflections for Slabs C1 to C7, respectively, excluding Slab C3. These values were considerably lower than those found for Moran RCC slabs. It was also noted that, with similar RCC moduli and subgrade stiffnesses and thicker slabs, Conley RCC slabs had larger maximum corner deflections than Moran RCC slabs. This was explained by the much lower LTE values found in the Conley RCC pavements.

From the cores taken from the four slabs, average slab thicknesses were 400, 370, 375, and 410 mm (15.8, 14.5, 14.8, and 16.2 in.) for Slabs M1, M2, M3, and M4, respectively. However, with the greatest-slab thickness, corner deflections in Slab M4 were greater than those in the other three slabs. Among other factors that affect pavement deflection under loading, an important element might be the fact that Slab M4 was adjacent to a stretch of asphalt pavement, and the corner was more or less free.

The same load could cause a very different deflection, depending on which side of a joint it was placed, as evidenced by those measured along the transverse joints of Slabs M1 and M2. A ratio of close to 2:1 was noted. Also, a nonlinear load-deflection characteristic existed in stations with the higher deflections (Stations 12.1 and 12.2 for Slab M1 and stations 12.1 to 12.4 for Slab M2). This might be an indication that certain pavement defects, such as voids under the slabs and cracks in the slabs, existed in the pavement system.

Maximum deflections along the transverse joints, longitudinal joints, and transverse lines 4.9 m (16 ft) from the joints are plotted in Figure 3 and Figure 4 for the Moran and Conley test slabs, re-

TABLE 2 Measured and Estimated Pavement Parameters

Slab No.	Load (kN)	Estimated ℓ (mm)	Estimated Subgrade Modulus (MPa/m)	Estimated RCC Modulus (MPa)	Measured RCC Modulus (MPa)	Difference (%)
	89	909*	104*	12,928*		
	156	1029	97	19,839		
M1	200	1018	97	18,829	17,741	11.0
	267	1022	103	20,414		
	Avg.	1023	99	19,694		
	89	892	128	18,944		
	165	872	132	17,887		
M2	205	923	123	20,960	19,399	-0.3
	280	870	145	19,547		
	Avg.	889	132	19,335		
	89	976	87	17,505		
	160	1009	84	19,154		
M3	200	1010	83	19,113	16,083	14.9
	267	960	96	18,129		
	Avg.	989	88	18,475		
	222	1027	158	24,580		
C1	289	1029	174	27,285	23,313	11.2
	Avg.	1028	166	25,932		
	200	1148	117	18,167		
C2	267	1199	119	21,951	20,867	-3.9
	Avg.	1173	118	20,059		
	205	1057	120	13,966		
C3*	280	1031	138	14,479	20,045	-29.0
	Avg.	1044	129	14,223		
	205	1119	110	19,252		
C4	267	1122	120	21,344	17,263	17.6
	Avg.	1120	115	20,298		
	200	1103	104	17,359		
C5	276	1101	124	20,571	19,428	-2.4
	Avg.	1102	114	18,965		
	200	1186	102	21,230		
C6	262	1046	126	15,890	18,067	2.7
	Avg.	1116	114	18,560		
	200	1080	126	18,936		
C7	276	1038	147	18,951	20,221	-6.3
	Avg.	1059	137	18,943		

Note: * excluded from further analyses
1 kN = 0.225 kip, 1 mm = 0.0394 in., 1 MPa = 0.145 ksi, 1 MPa/m = 3.684 pci

spectively. Deflections were expressed as percentages of the maximum values for that particular series. For example, maximum values would occur under edge loading when loads were moving along the transverse lines. The distance from the corner or the joint was expressed in units of l (radius of relative stiffness). It was observed that, except for deflections along the transverse joint in Slab M1, the deflection ratio decreased as the load moved inward from the joints. The rate of deflection ratio decrease also decreased with increasing distance from the corner or joint. Excluding data from Slab M1 transverse joint loads, regression lines were developed for these three series of deflections. Although showing some scatter, with the coefficient of determination in the range of between 0.700 and 0.900, a decent relationship between the deflection ratio and the distance from the joint exists. It is also interesting to note that the three regression lines for Moran test slabs were very close to one another, suggesting that the relationship may be unique, regardless of the position of deflection measurements. However, the three regression lines were different for Conley RCC test sections.

Deflection measurements for LTE determination were made on Slabs M1 to M3, Slabs C1 to C7, and at other locations throughout the two RCC sites. Tests were conducted at 51 locations at Moran RCC sites and 66 locations at Conley RCC sites. The joints tested included construction joints and cracks in both longitudinal and transverse directions. LTE measurements were taken from both sides of the joints, and four load levels were used.

Earlier studies by the U.S. Army Corps of Engineers have revealed widely scattered LTE values on various RCC pavements. Therefore, joints with zero load transfer were generally assumed for RCC pavement thickness design. Similar variability in LTE values was observed in Moran and Conley RCC pavements. For Moran test sections the LTE values ranged from 18 to 87 percent, with an average value of 49 percent and a coefficient of variation of 40 per-

TABLE 3 Adjusted Modulus of Subgrade Reaction

Slab No.	Load (kN)	Adjusted Modulus of Subgrade Reaction (MPa/m): Distance up to 0.9 m from load	Distance of 0.9 to 1.5 m from load	Distance above 1.5 m from load
M1	156	143	97	67
	200	145	97	66
	267	156	103	70
	Average	148	99	68
M2	89	202	128	83
	165	213	132	91
	205	193	123	87
	280	244	145	101
	Average	213	132	90
M3	89	133	87	61
	160	122	84	57
	200	123	83	58
	267	152	96	68
	Average	132	88	61
C1	222	203	158	131
	289	225	174	146
	Avg.	214	166	138
C2	200	150	117	98
	267	148	119	96
	Avg.	149	118	97
C4	205	138	110	89
	267	153	120	97
	Avg.	146	115	93
C5	200	128	104	86
	276	157	124	104
	Avg.	143	114	95
C6	200	124	102	82
	262	160	126	102
	Avg.	142	114	92
C7	200	161	126	98
	276	192	147	113
	Avg.	176	137	106

1 m = 3.28 ft, 1 kN = 0.225 kips, 1 MPa/m = 3.684 pci

cent. Similar LTE values were observed for both the shrinkage cracks and the construction joints. The average LTE was 48 percent for cracks, with a coefficient of variation of 44 percent, and 51 percent for construction joints, with a coefficient of variation of 37 percent. The LTE values for Conley pavements showed an even larger variation compared with those for the Moran slabs. They ranged from 12 to 100 percent, with the majority being less than 50 percent. The average LTE was 36 percent, with a coefficient of variation of 56 percent.

No trend in the LTE values at different points along a joint could be found. Some had similar LTE values across the joint, such as the transverse joint of Slab C2, and some showed different behaviors, such as the transverse joint of Slab C7. Therefore, it is essential to specify the locations of measurement when dealing with LTE determinations.

Analysis of Laboratory Test Data

Three cores were taken from each of Slabs M1 to M4, and two cores were taken from each of Slabs C1 to C7. All of the cores had a nominal diameter of 95 mm (3.75 in.). The core thicknesses were close to one another within each slab except for Slab C1, which had a 140-mm (5.5-in.) difference in length. However, some variations in thickness were noted between slabs. For Moran test sections the layers were found to be fully bonded for most of the cores, whereas the layers of half of the Conley cores were found to be separated. The interfaces between lifts could not be easily identified for the bonded specimens. All of the cores were cut into either two or three pieces after initial examination.

Direct shear tests were conducted on four cores before they were cut. The shear strengths were 6240, 2655, 1758, and 2069 kPa (905,

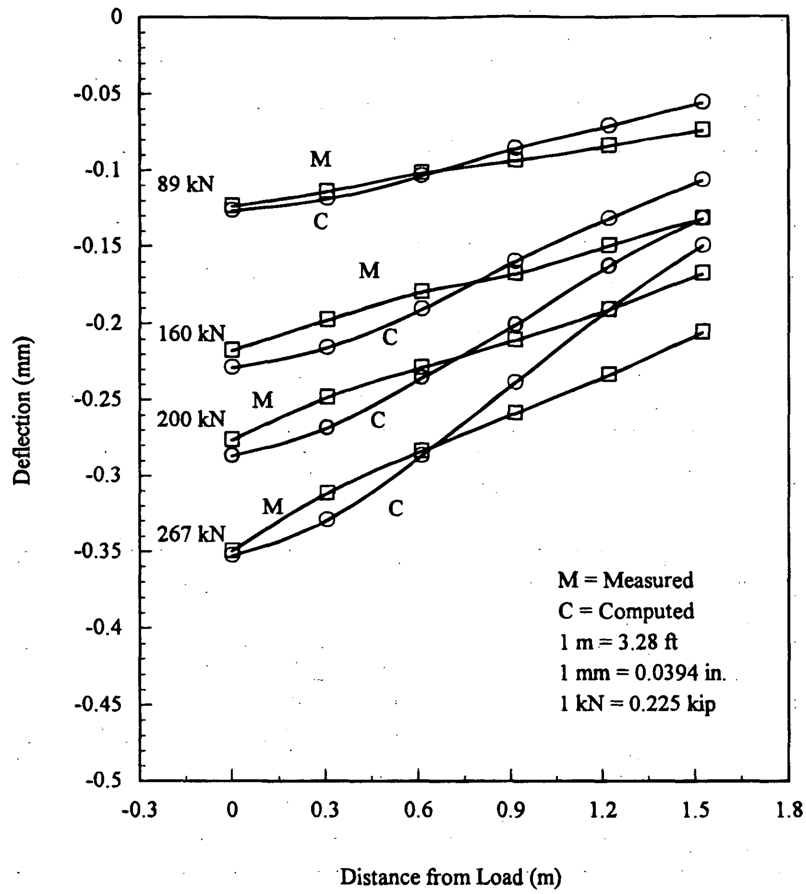

FIGURE 2 Typical comparison between measured and computed deflections.

385, 255, and 300 lb/in.^{2}) for these specimens, and the failure planes were coincident with the observed interfaces. It has been reported that for conventional normal-weight concrete constructed in a single lift, shear strength can vary from 35 to 80 percent of the compressive strength (*13*), or from 50 to 80 percent of the compressive strength (*14*). The RCC cores showed low shear strengths at the interfaces, ranging from 5 to 16 percent of the compressive strengths. Therefore, it is reasonable to hypothesize for design purposes that multi-layer construction of RCC pavements will result in weakened horizontal planes between lifts.

All prepared specimens were subjected to tests to determine the RCC modulus of elasticity and unit weight. Compressive strength and split tensile strength were also determined for selected samples. Table 4 lists all of the laboratory test results. It can be observed that the values of compressive strength, split tensile strength, and unit weight are comparable to those of conventional PCC, whereas the RCC moduli of elasticity were found to be less than the expected moduli of elasticity of PCC, with all other factors being equal.

It has been well documented that density (unit weight) is an important factor in affecting the strength of RCC. A small reduction in density will result in a significant decrease in strength (*15–17*). In this research an attempt was made to study the relationship between density and RCC strengths and elastic moduli. By using regression techniques the following equations were developed:

$$f_c' = 464 \cdot W - 63{,}189 \quad (1)$$
$R^2 = 0.6311$
$n = 21$

$$ST = 36.6 \cdot W - 4756 \quad (2)$$
$R^2 = 0.6559$
$n = 50$

where

f_c' = compressive strength (lb/in.^{2}),
W = density (pcf),
ST = split tensile strength (lb/in.^{2}), and
n = number of datum points.

Two datum points from Sample M1-2 at the middle and Sample M4-1 at the top were not used in deriving Equation 1 because of extreme values. Predicted compressive strengths and split tensile strengths ranged from 12.2 to 44.2 MPa (1,771 to 6,411 lb/in.^{2}) and from 2.5 to 5.1 MPa (368 to 734 lb/in.^{2}), respectively, for RCC, with densities being between 2240 and 2400 kg/m^3 (140 and 150 lb/ft^3). These values were comparable to those of normal-weight concrete.

Following the form of the American Concrete Institute (ACI) equation, the RCC modulus of elasticity can be estimated from its compressive strength and density by the following equation:

$$E_r = 21.6 \cdot W^{1.5} \cdot \sqrt{f_c'} \quad (3)$$
$n = 23$

where E_r is the modulus of elasticity of RCC (lb/in.^{2}).

Compared with the ACI equation for conventional PCC, with strength and density being equal, the modulus of elasticity of the

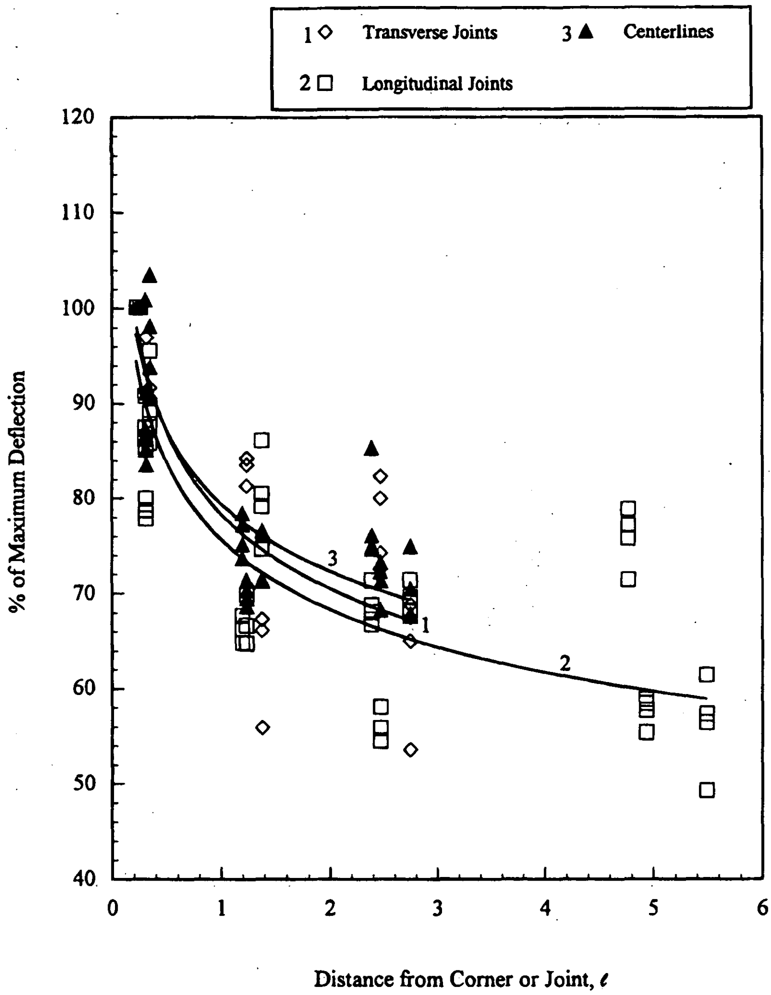

FIGURE 3 Deflection ratio versus distance, Moran Terminal.

RCC at the Moran and Conley Terminals was about 65 percent of that of regular PCC. Since the strength of RCC is comparable to that of PCC and a higher concrete elastic modulus will result in a higher stress in concrete slabs, the lower modulus of elasticity can be beneficial to RCC pavement performance.

Structural Evaluation of RCC Test Slabs

By using the estimated pavement parameters from the HWD deflection data and with the aid of the program JSLAB, the structural adequacy of the RCC slabs was assessed in two ways. In the first method the maximum stresses in the slabs caused by the anticipated traffic loads were computed. The computed maximum stresses were then used along with the RCC strengths and Miner's hypothesis to determine the number of loads that the pavements could take before failure. This number could be used to check if the design thickness was adequate. All of the loads were applied at the slab edge.

The second method, known as critical condition analysis, was based on the concept that a pavement system might fail under a single application of the worst possible condition rather than under fatigue. It had been established that a load applied at the slab edge with a high positive thermal gradient (the top warmer than the bottom) in the slab would induce the highest stress in the pavement slab. Furthermore, a free edge condition (no load transfer) was assumed in the analysis. It should be noted that instead of using variable moduli of subgrade reaction, the highest value was used for each slab because the analytical computer program could not handle both temperature gradient and variable modulus of subgrade reaction at the same time.

The tire load of a fully loaded Letro-Porter at slab edge and a temperature differential of +11°C (+20°F) were used in representing the worst possible condition. The use of +11°C (+20°F) was arbitrary since few data were available. However, with the thickness ranging from 380 to 510 mm (15 to 20 in.) and on the basis of experience with PCC pavements, it was believed that this number could realistically be achieved during summer days.

In their study Tayabji and Okamoto (*10*) developed relationships between RCC flexural strength and compressive strength for four different RCC mixes. The equations followed the form of the general ACI equation, $f_r' = C \cdot \sqrt{f_c'}$. The constant C ranged between 9.4 and 10.8 for the four RCC mixes, with an average value of 9.9, which was used in estimating the flexural strengths of the Moran and Conley RCC pavements. The fatigue equation developed in the same study was also used to predict the allowable number of load repetitions. These equations are given below:

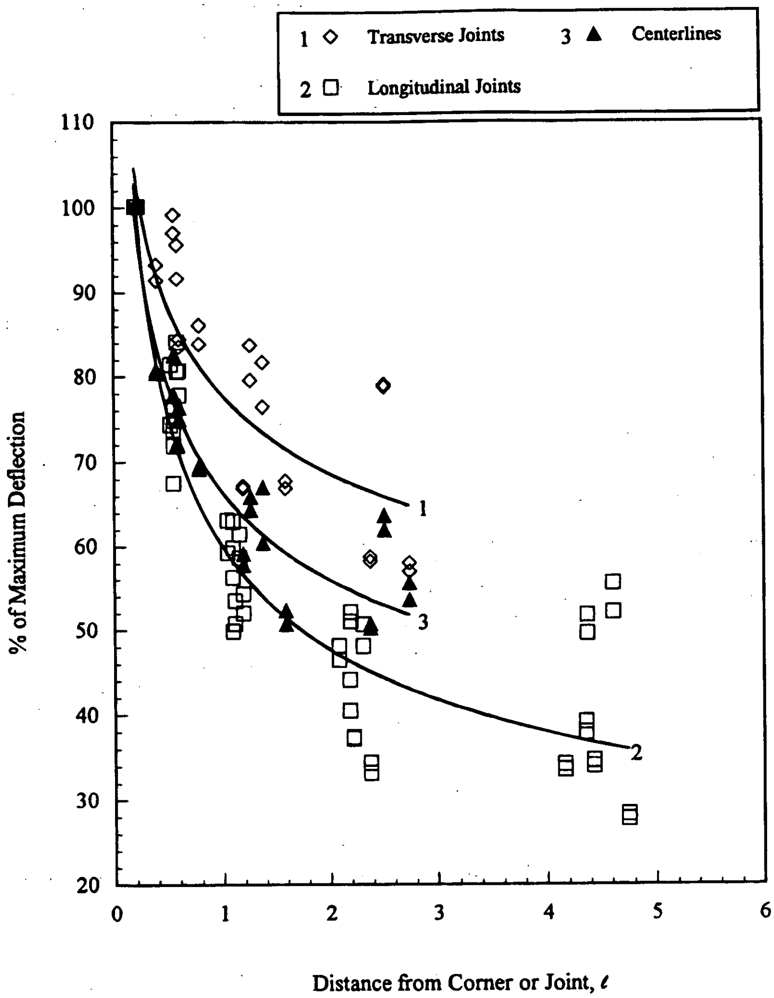

FIGURE 4 Deflection ratio versus distance, Conley Terminal.

$$f_r' = 9.9 \cdot \sqrt{f_c'} \quad (4)$$

$$SR = 118.31 - 10.73 \cdot \log(N) \quad (5)$$

where

f_r' = flexural strength (lb/in.²),
f_c' = compressive strength, (lb/in.²),
SR = stress to strength ratio (percent), and
N = allowable number of load repetitions.

The results of the structural evaluation are given in Table 5. The evaluation was not conducted for Slabs M4 and C1 because of insufficient data and for Slab C3 because of large errors in the estimated RCC elastic modulus. As observed, without considering the temperature effect, the stresses caused by the load ranged from 2.8 to 3.1 MPa (408 to 455 lb/in.²) for the Moran test sections and from 2.0 to 2.2 MPa (288 to 323 psi) for the Conley RCC. The stress-to-strength ratios were 55, 63, and 72 percent, resulting in allowable load repetitions of 835,933, 151,822, and 20,225 for Slabs M1, M2, and M3, respectively. For a design life of 20 years, with operation 365 days a year, Slab M3 could take maximum load repetitions of about 3 times a day, whereas Slab M1 would be allowed to take more than 114 load repetitions each day at any given point. Although the structural performance of Slab M3 can be expected to be marginal, Slab M1 can be expected to last a long time. The large variations in the structural performance were mainly due to the variation observed in the actual strength of the RCC pavement and, to a lesser extent, in the thickness of the pavement.

The stress-to-strength ratio ranged from 32 to 51 percent for Conley RCC test sections. The allowable numbers of load repetitions were mostly unlimited except for Slab C2, which could take about 2 million load repetitions before failure. Therefore, the thickness design for Conley RCC slabs could be regarded as conservative.

Under the critical thermal-loading condition, stress-to-strength ratios were 80, 89, and 100 percent for Slabs M1, M2, and M3, respectively. A ratio of 100 percent would cause the slab to crack. However, this condition would probably rarely happen considering the assumption of free edge loading and a high thermal gradient in the slab. It can be predicted that Slabs M1 and M2 will perform better than Slab M3. With the stress-to-strength ratio ranging from 52 to 79 percent, it can be inferred that the Conley RCC test sections are structurally adequate. Results of the evaluation as well as laboratory tests have indicated great variability in the performances of RCC pavements and the material strengths of RCC pavements.

TABLE 4 Results of Laboratory Tests on RCC Specimens

Sample No.	Modulus of Elasticity (MPa)			Compressive Strength (kPa)			Split Tensile Strength (kPa)			Unit Weight (kg/m^3)		
	Top	Middle	Bottom	Top	Middle	Bottom	Top	Middle	Bottom	Top	Middle	Bottom
M1 – 1	16,475	20,787	17,114	*****	*****	*****	3,682	5,688	5,923	2295	2398	2387
M1 – 2	18,143	13,651	19,196	39,550	31,172	*****	*****	*****	867	2366	2409	2430
M1 – 3	17,322	18,699	18,281	*****	*****	46,569	4,358	5,509	*****	2332	2412	2414
M2 – 1	16,231	24,915	20,704	34,572	*****	35,895	*****	5,385	*****	2379	2390	2335
M2 – 2	18,185	19,362	15,044	*****	33,027	*****	4,661	*****	3,406	2363	2352	2270
M2 – 3	19,860	19,971	20,322	*****	*****	*****	4,827	5,351	4,496	2372	2395	2361
M3 – 1	19,278	17,699	15,457	*****	*****	*****	3,827	3,668	3,503	2289	2332	2244
M3 – 2	16,595	14,831	12,674	37,826	24,932	*****	*****	*****	2,006	2398	2297	2207
M3 – 3	16,522	18,298	13,394	*****	*****	21,306	3,330	4,068	*****	2324	2356	2311
M4 – 1	18,538	24,176	17,947	46,265	*****	*****	*****	6,316	5,006	2321	2430	2417
M4 – 2	17,328	22,914	20,306	*****	46,141	*****	3,158	*****	4,882	2281	2403	2379
M4 – 3	19,764	22,180	19,751	*****	*****	45,004	3,923	4,599	*****	2319	2379	2382
C1 – 1	20,864	*****	20,665	*****	*****	*****	540	*****	3,944	2356	*****	2360
C1 – 2	24,262	*****	27,459	*****	*****	*****	681	*****	3,647	2371	*****	2319
C2 – 1	16,354	19,309	21,111	*****	*****	*****	4,268	4,323	5,309	2382	2347	2408
C2 – 2	16,744	19,654	32,033	*****	*****	57,932	5,226	3,620	*****	2374	2372	2409
C3 – 1	17,066	20,252	17,208	*****	*****	*****	3,689	5,192	4,468	2366	2401	2385
C3 – 2	18,195	23,523	24,029	*****	*****	*****	4,799	5,109	5,502	2356	2392	2388
C4 – 1	19,312	15,143	17,175	35,123	21,437	29,069	*****	*****	*****	2356	2315	2308
C4 – 2	17,697	17,500	16,753	*****	*****	*****	4,378	3,441	3,434	2358	2326	2313
C5 – 1	22,439	19,041	19,828	41,480	44,328	40,812	*****	*****	*****	2352	2395	2374
C5 – 2	18,474	19,132	17,655	*****	*****	*****	4,061	5,130	3,275	2361	2401	2368
C6 – 1	14,147	20,868	17,837	*****	*****	*****	2,661	4,551	4,985	2310	2352	2339
C6 – 2	12,514	*****	24,971	22,471	*****	36,709	*****	*****	*****	2316	*****	2345
C7 – 1	11,678	*****	22,152	*****	*****	*****	4,378	*****	4,103	2390	*****	2396
C7 – 2	22,655	*****	24,399	32,462	*****	33,089	*****	*****	*****	2390	*****	2401

Note: ***** Not available
1 MPa = 0.145 ksi, 1kPa = 0.145 psi, 1 kg/m^3 = 0.0624 pcf

TABLE 5 Results of Structural Evaluation of RCC Test Sections

Slab No.	Load Only				Load & Thermal Gradient		
	Flexural Stress (kPa)	Flexural Strength (kPa)	Percent of Strength	Allowable Repetition, N	Flexural Stress (kPa)	Flexural Strength (kPa)	Percent of Strength
M1	2,813	5,137	55	835,933	4,096	5,137	80
M2	3,027	4,827	63	151,822	4,296	4,827	89
M3	3,137	4,351	72	20,225	4,351	4,351	100
C2	1,986	6,254	32	Unlimited	3,227	6,254	52
C4	2,227	4,392	51	1,997,217	3,489	4,392	79
C5	2,227	5,344	42	Unlimited	3,413	5,344	64
C6	2,151	4,475	48	Unlimited	3,303	4,475	74
C7	2,137	4,709	45	Unlimited	3,323	4,709	71

1 kPa = 0.145 psi

Therefore, it is suggested that an additional factor of safety be used in RCC pavement design.

CONCLUSIONS

From the analyses of the nondestructive load testing conducted on Moran and Conley Terminal RCC pavements, the following conclusions are drawn.

1. Estimated in situ pavement parameters, obtained by using the program ECOPP and the HWD deflection data, were considered to be reasonably accurate when compared with the values measured in the laboratory.
2. Under loading RCC pavements showed behavior similar to those of conventional concrete pavements. However, widely scattered LTE values across joints or cracks were observed. They ranged from 18 to 87 percent, with an average of 49 percent and a coefficient of variation of 40 percent, for Moran RCC slabs and from 12 to 100 percent, with an average of 36 percent and a coefficient of variation of 56 percent, for Conley RCC slabs.
3. The LTE values calculated by using deflection measurements at different locations along a joint may be quite different. Therefore, it is important to specify the exact locations in measuring and reporting LTE measurements.
4. For Moran and Conley test sections, the strengths and densities of RCC pavements were in the same range as those of PCC pavements, whereas RCC moduli of elasticity were found to be only about 65 percent of PCC moduli of elasticity, with all other factors being equal.
5. The strength of RCC was highly dependent on its density. A small reduction in density would reduce the strength considerably.
6. The RCC pavement design seemed to be adequate from the structural evaluation. However, because of the large variations in the material properties of RCC and field performance, a higher factor of safety may be needed in designing RCC pavements than in designing conventional concrete pavements.

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