Dynamic Response of Rigid Pavement Joints

THEODOR KRAUTHAMMER AND LUCIO PALMIERI

A recent experimental study on the relationship between aggregate interlock shear transfer across a concrete interface and its dynamic response frequency is described. The analysis of the data recorded during the tests was performed in both the time and the frequency domains. The results demonstrate a clear relationship between the internal aggregate interlock conditions of the concrete interface and its response frequency, and the study may serve as a basis for an innovative testing approach for joints in rigid pavements.

Pavement maintenance for ensuring acceptable serviceability conditions requires the assessment of a pavement's internal conditions. However, the determination of such conditions in extensively damaged pavement can be difficult. In rigid pavements, one of the most critical areas is the joint where spalling of concrete and cracking and pumping of the base may occur. These phenomena can adversely affect the load transfer across pavement joints and the pavement's behavior. In the last decades, significant efforts have been undertaken to develop nondestructive testing techniques for pavement evaluation. Most of these techniques, however, provide information on pavement deflection, an indirect parameter largely used for pavement structural evaluation. Furthermore, deflections depend on the modality of loading and on its magnitude, and one of the major problems is to identify an experimental loading mode representative of traffic loads.

The objective of this paper is to describe a recent experimental study on the assessment of shear transfer conditions by aggregate interlock in a rigid pavement joint on the basis of the frequency analysis of its dynamic response (*1,2*). The analysis of the data recorded during the tests was performed in both the time and the frequency domains. The correlation between the results obtained in these two domains highlighted a unique relationship between the shear transfer ability and the dynamic response frequency.

BACKGROUND

Joints are locations where geometric discontinuities are introduced into the pavement slab, and the provision of load transfer across a joint is one of the main concerns in concrete pavement design. In undowelled joints, load transfer is provided by shear forces across an undowelled discontinuity between adjacent edges in the form of aggregate interlock. In reinforced concrete pavements the reinforcing steel is expected to resist the opening of undesired cracks to ensure adequate aggregate interlock and to provide dowel action across discontinuities in the pavement. These mechanisms have been described in a report by American Concrete Institute–American Society of Civil Engineers Committee 426 (*3*) and in well-known books on reinforced concrete (*4,5*). Furthermore, it is known that the development of good aggregate interlock can greatly prevent the development of critical edge stresses (*6,7*).

Application of shear to planes of discontinuity (cracks, interfaces between different materials, interfaces between concretes cast at different times) causes relative slip between the two sides of the discontinuity. As reported by Park and Paulay and by MacGregor (*4,5*), if reinforcement is present, the shear is transferred across the interface mainly by the following two mechanisms:

1. Under the shear action the surfaces tend to separate or to slip, or both; in both cases the reinforcement is subjected to tension, and for equilibrium the concrete is subjected to compression. Consequent to these compressive stresses, friction develops in the interface.
2. Shear is also transferred by aggregate interlock between the particles of concrete and by dowel action of the reinforcement crossing the surface.

According to Mattock and Hawkins (*8*), the shear strength V_n when shear friction reinforcement is perpendicular to the shear plane can be evaluated from

$$V_n = 0.8A_{vf}f_y + A_cK_1 \tag{1}$$

where

A_{vf} = area of reinforcement crossing the surface,
A_c = area of concrete surface resisting the friction,
K_1 = constant equal to 400 for normal-weight concrete, and
f_y = reinforcement's yield strength.

The first term in Equation 1 represents mechanism 1 (friction) with a coefficient of friction for concrete sliding on concrete of 0.8, and the second term represents mechanism 2. Furthermore, the first term in Equation 1 should be notified if the two concrete surfaces are pressed against each other by an externally applied in-plane force, and such a modification could be based on the Coulomb friction theory. This shear force is expected to control the load transfer across joints in rigid pavements.

T. Krauthammer, Department of Civil Engineering, 212 Sackett Building, Penn State University, University Park, Pa. 16802. L. Palmieri, Department of Civil and Mineral Engineering, University of Minnesota, 500 Pillsbury Drive S.E., Minneapolis, Minn. 55455.

Current joint assessment approaches employ the concept of joint efficiency for classification of its internal condition. Load transfer efficiency (or joint efficiency) is defined as the ratio of the deflection of the unloaded side of the joint divided by the deflection of the loaded side of the joint (for convenience this ratio is usually expressed as a percentage), and it is related to the joint shear transfer capacity. From the study by Foxworthy (*9*) it has been shown that the load transfer efficiency of a joint is closely related to the stresses that are developed on the bottom of the slab and therefore to the slab's performance under loading. Tabataie and Barenberg (*10*) showed that the load transfer efficiency across a joint affects the maximum stresses in the slab, especially under edge and corner loading conditions.

Since the stress transmitted to the subgrade, assuming linear behavior, is the slab deflection times a base stiffness K, the subgrade stress will be affected in proportion to the deflection. The critical point to note is that just a small reduction from a full joint efficiency may result in a large change in the stress ratio, and therefore significant benefits correspond to designing joints with high load transfer efficiencies. The joint efficiency depends on several factors, such as temperature, moisture, and frost, but, according to Foxworthy (*9*), not on the load magnitude.

Nondestructive Evaluation Techniques for Pavements

Nondestructive techniques are methods for evaluating the characteristics of a system without harming it. In this case, the goal is to determine the strength and integrity of the pavement, usually by employing deflection measurements. The deflection of a rigid pavement under loading is generally due to comparison of the base rather then to compression of the pavement layers (*6,7*). Pavement deflections depend on pavement properties and on the loading type (e.g., static, dynamic) and magnitude. The ideal testing procedure should closely represent a design moving load. According to a classification by Moore et al. (*11*), the four major categories of nondestructive structural evaluation of pavements are static deflection, steady-state deflection, impact load response, and wave propagation techniques. The impact load response techniques have several advantages over the others (*6,7*), and this approach was employed for the present study.

In impact load response techniques, a transient load is applied to the pavement, and the response, usually in terms of displacement, is measured. The short-duration loading is generated by a weight dropped on a plate that is placed on the pavement surface, but according to Moore et al. (*11*), the duration of the pulse should not exceed 1 msec for the loading to be considered transient. This is because the rise time, defined as the time the pavement needs to deflect from 10 to 90 percent of its maximum deflection, can vary between 3 to 6 msec. Furthermore, the response to longer loadings will contain not only information on short-duration pavement response, but also information on the longer-frequency characteristics of the system and disturbances from wave reflections. It is difficult to obtain such short pulses in the field, and the devices used for impact load tests usually have pulses that last 20 msec or longer.

One such testing device is the falling weight deflectometer (FWD), which consists of a large mass that is dropped vertically on a plate resting on the pavement surface while a spring-damping system is interposed between the mass and the plate. The mass, dimensions of the plate, and drop height vary, depending on different versions of the device. For example, the Phoenix FWD has a 330-lb weight, it is dropped from a height of 15.7 in. on an 11.8-in.-diameter circular plate, and the pavement's deflection is measured with a linear variable differential transformer (LVDT). The corresponding pulse has a duration of about 26 msec, and the magnitude of the peak load is about 5.5 tons.

Different types of FWD devices generate pulses with different durations, but always in the interval from 20 to 40 msec. The theoretical accelerations transmitted to the pavement by these devices are of the order of 10 to 30 g (where g is the acceleration of gravity). For example, the accelerations measured in the field by Hoffman and Thompson (*12*) were about 4 g, and this difference was probably due to the FWD-pavement interaction and to the rubber mat interposed between the loading plate and pavement surface. However, even a 4 g peak acceleration is about 10 times higher than those due to traffic, as pointed out by Sebaaly et al. (*13*), who also noted that the duration of typical pulses from traffic loading is of the order of several hundred milliseconds. Despite these discrepancies, the deflections measured by Hoffman and Thompson (*12*) were consistent with the ones due to traffic; however, better results are obtained by using velocity transducers (geophones) instead of LVDTs. This is because geophones, like accelerometers, do not need an immovable reference system as required by an LVDT.

It is clear that all impact load response techniques are based on measuring the dynamic response of a pavement under a short-duration load pulse. Furthermore, dynamic analyses can be performed both in the time and the frequency domains, and the information obtained by one approach supplements the other. The traditional dynamic analysis has been based on time domain methods (*14*); however, much can be learned from frequency domain analysis. Data acquired during tests represent the time history of a certain variable. The Fourier transform is one of the tools used to study a function's characteristics in the frequency domain, and the analysis in the frequency domain was done using the fast Fourier transform (FFT) techniques, as discussed by Oppenheim and Schafer (*15*). The FFT operation was applied through the scientific software Asystant+ (*16*). In fact, what was actually done in this study was the power spectrum operation, defined as the square magnitude of the FFT.

Structural Response Under Impulsive Loads

Structures behave differently under dynamic loads than when subjected to static forces. However, not all types of dynamic loads produce the same modes of behavior, as discussed by Clough and Penzien (*14*) and Biggs (*17*), who showed that impulsive loads can excite specific behavioral modes that do not exist under slower loading conditions. Krauthammer et al. (*18,19*) showed that under very short impulsive loads there is a complete separation between flexural and shear responses. These findings were confirmed and carefully examined also by Assadi-Lamouki and Krauthammer (*20*), who found that the interaction between structural response mechanisms depends on their corresponding natural frequencies.

Furthermore, it has been found by Krauthammer and others (*18–20*) that such time separation between flexural and shear response modes indicates hardly any influence of flexural behavior on shear behavior.

For example, a reinforced concrete slab with the dimensions 60 × 30 × 6 in. has a fundamental (flexural) frequency of about 0.0737 Hz, and a 30 × 30 × 6 in. slab has a fundamental frequency of about 0.1178 Hz (these frequencies correspond to periods of about 13.5 and 8.5 sec, respectively). Impulsive loads, however, can excite shear response modes in the frequency range of several hundred, or even thousands, of hertz (i.e., corresponding to periods that could be less than 1 msec). Under such conditions, the structure will exhibit its thickness shear responses first, and the flexural response modes may appear much later. Obviously, one can perform experiments in which the early shear response is captured by high-speed data acquisition systems, and any possible flexural response would not affect that behavior since it only appears much later. It was shown earlier that typical FWD devices use relatively heavy weights dropped from a low height onto an elastic layer that is placed over the pavement surface. Increasing the drop height and reducing the flexibility of the elastic layer and the weight enable one to obtain shorter load pulses with comparable magnitudes. The experimental approach adopted for this study is based on the early time shear response of concrete slabs.

APPROACH OF THE STUDY

The present study (*1*) was aimed at developing an experimental procedure for evaluating aggregate interlock shear transfer capabilities across portland cement concrete (PCC) pavement joints. Furthermore, on the basis of the separation between flexure and shear response modes, as discussed above, only the early time (i.e., shear) behavior would be of interest for this method, and there is no need to study the flexural behavior (i.e., the moment transfer) also, which has no effect on the early response. Krauthammer and Western (*21*) showed that the shear transfer across a joint in rigid pavements can be described accurately by the relationship between shear stress and shear slip. A deterioration in the joint shear transfer capability was correlated to a reduction in the joint shear stiffness. Such a reduction in the joint's stiffness should cause a decrease in the shear response frequency, as discussed by Clough and Penzien and Biggs (*14,17*).

This study was based on the anticipated finding of a relationship between the frequency response of a joint to dynamic loading and the joint shear transfer capability. If a relationship between the internal aggregate interlock shear transfer conditions of the joint and its response frequencies exists, it would be possible, by measuring a corresponding frequency shift in situ, to determine the joint's internal condition in a unique manner. Such information would then be used to decide on the required corrective measures, which would enhance roadway management procedures. The experimental approach has been described elsewhere (*2*), and a brief summary is presented here.

The following two reinforced concrete slab systems were employed in modeling the pavement-joint configuration in this study, as shown in Figure 1:

1. System 1: One concrete slab 60 in. long, 30 in. wide, and 6 in. deep.
2. System 2: Two concrete slabs each 30 in. long, 30 in. wide, and 6 in. deep.

Although these systems were not intended to model a full-scale pavement joint, they contain all material and geometric features to provide meaningful information on aggregate interlock across a concrete interface. System 1 represents the ideal case of full shear transfer across an imaginary joint, and the two slabs of System 2 represent a realistic concrete-to-concrete interface in a pavement. The two systems were placed

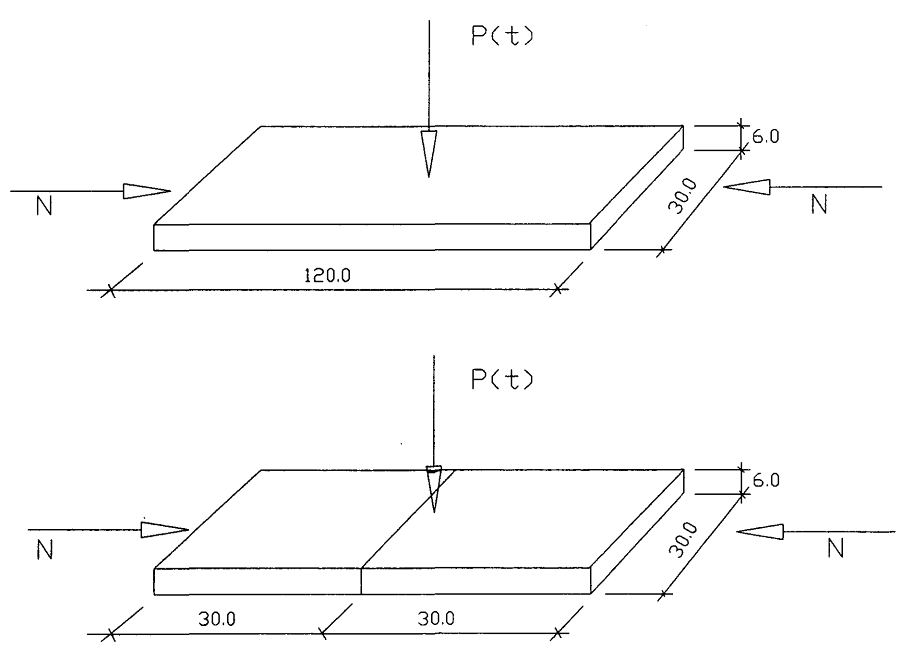

FIGURE 1 Slabs and force systems.

on a sand bed 1 ft deep to simulate an underlying base material.

The simulation of different conditions of joint shear transfer in System 2 was done by applying to the system an in-plane compressive force N, as shown in Figure 1. The application of this force was realized by means of two 1-in.-diameter steel rods connected to steel angles placed at the ends of the slabs. When the nuts at the end of the rods were torqued, tensile forces were induced in the rods and a corresponding compressive force was induced in the plane of the slabs. The axial forces in the rods were calculated from the measured strains obtained from two strain gages that were mounted diametrically opposite, and their readings were averaged to remove possible bending effects. The average strain in each rod was multiplied by the steel modulus of elasticity and by the cross-sectional area to compute the axial force.

Under loading, when the vertical force $F(t)$ is applied, the two slabs are forced to move relative to each other, and a frictional force T, as defined in Equation 1, develops at the interface opposing this relative movement. Varying the values of the force N, the values of T change accordingly, as will the total resistant forces acting on the two sides of the interface. Neglecting the friction between the slabs and the sand, this force will be close to $F(t) - T$ on the loaded side of the joint, whereas on the unloaded side it will be T. Changing the magnitude of these resultant forces causes a difference in the corresponding displacements of the two sides of the joint; hence, the value of joint efficiency can be computed. This approach permits one to vary the value of N for simulating different shear transfer conditions and corresponding joint efficiencies.

The dynamic load was generated by a 30-lb weight dropped from a height of 3 ft. The impulsive force obtained in this manner had a recorded maximum amplitude of 22,500 lb and a duration of about 1.6 msec. These pulse characteristics are close to the limit of 1 msec, as recommended by Moore et al. (*11*), and they have peak force values comparable with in-service loads or FWD tests. The impact load was uniformly distributed over a steel plate with an area of 144 in.^{2} to prevent cracking and breaking of the concrete. The plate was positioned along the major axis of symmetry of the slab 3 in. from the joint and firmly attached to the slab by four steel bolts to prevent rebound. Two rubber pads were interposed between the plate and the concrete slab and between the plate and the weight to prevent steel-to-steel high-acceleration impact, to keep the accelerations under the maximum value of 500 g, and to obtain a more even distribution of the load from the steel plate onto the concrete slab. This approach ensured that the same impact conditions would exist for all tests.

Experimental data consisted of acceleration-time histories obtained from accelerometers glued to the reinforced concrete slabs. Two accelerometers were positioned 2.5 in. from the joint along the major axis of symmetry of the slab on both sides of the joint, and they were connected to the data acquisition system through a channel coupler. Data acquisition was performed at an effective sampling rate of 500 kHz per channel. Data were acquired for 10 msec, and the number of samples acquired from each channel was 5,000. This duration was sufficient to capture the vibrations of the systems under the main impact. As noted earlier, the flexural response of the slabs could appear between about 8.5 to 13.5 sec later, well after the data had been acquired. The experimental data were then transferred for analysis in both the time and frequency domains by the scientific software Asystant+ (*16*).

Tests on System 1

A set of eight tests, marked B1 through B8, was conducted without applying in-plane compression to the slab through the rods, and they represented an ideal condition of full shear transfer. The data were acceleration-time histories on both sides of the joint in the positions previously described. However, any comparison between the results from System 1 and the results from System 2 must be made only at parity of conditions. Therefore, it was necessary to eliminate all influences on the results by any test parameter except the joint shear transfer. Preliminary experiments were conducted and it was determined that neither large in-plane forces nor possible background noise had an effect on this experiment.

Tests on System 2

System 2 represented variable shear transfer conditions across a joint induced by externally applied in-plane forces to the pavement slabs, as described above. Different values of in-plane compression were applied to the slabs, tests were conducted, and the corresponding values of joint efficiency were calculated. On the basis of the data obtained, and according to the classification by Krauthammer and Western (*21*), values of axial force were selected to represent the following three joint conditions: a new joint, a deteriorated joint, and a dead joint, as shown below:

	Joint Efficiency
New joint	≥ 0.9
Deteriorated joint	≈ 0.6
Dead joint	<0.4

Six values of the force N were selected to be applied in each rod to represent the different joint conditions and, correspondingly, six sets of three tests each were conducted on System 2. Every test was marked by two letters followed by a number from 1 to 3, and their corresponding in-plane forces are as follows:

Test Set	*In-Plane Force, Each Rod (lb)*
ST	2,000
SU	4,000
SV	5,000
SP	6,000
SQ	8,800
SR	9,500

RESULTS AND DISCUSSION

Experimental data consisted of acceleration-time histories obtained from the accelerometers attached to the reinforced concrete slabs in the positions previously described. The displacement-time histories at those points were obtained by integrating twice the acceleration records. On the basis of the frequency analysis of the acceleration data, it was decided to concentrate on the frequency range between 0 and 5,000 Hz. The results obtained for the two systems under consideration are discussed next.

Results for System 1

As described previously, a set of preliminary tests was conducted on System 1 to evaluate the effort of in-plane forces on the system's response frequency, and it was noted that the frequencies were the same as those obtained without the application of in-plane forces. From this observation it was decided to consider the results for the case without in-plane forces as the main set of tests, and the corresponding data would be compared with that from the tests on System 2.

The assumption that System 1 represents an ideal condition of full shear transfer capability is confirmed by the test results, and the initial assumption of a joint efficiency value was 0.9, as shown earlier. The experimental values obtained for the maximum displacements, the time at which they occur, and the corresponding joint efficiency are given in Table 1.

It is noted that for six of the eight cases these values are very close to 0.9. However, a value of 0.628 was obtained for case B3 and a value of 1.132 for case B6, most probably because of poor adherence between accelerometers and the slab. It was noted earlier that the joint efficiency ratio is obtained when the peak deflection of the unloaded side of the joint is divided by the peak deflection of the loaded side. Ratios considerably lower or higher than unity are possible since either one of the accelerometers may not have been adequately bonded to the surface. From Table 1 it is noted that the maximum displacements for System 1 occur at the same time on both sides of the joint. In general for every test, all time histories of these displacements almost overlap: the two sides of the imaginary joint move at the same time and with displacements of the same amplitude. This indicates that the load is properly transferred from one side of the ideal joint to the other, as expected. The data were used to obtain a description of the response in the frequency domain for all cases. It should be noted again that the flexural (i.e., bending) response would appear much later, well after the test is over.

Results for System 2

System 2 represents the variable joint conditions through the applications of different values of in-plane force N. Table 2 contains the average values of maximum displacements, the time at which they occur, and the corresponding joint efficiency values (i.e., the ratio of the peak displacement of the unloaded side to the peak displacement of the loaded side). The selection of in-plane compression, reported in the previous section, as representative of variable shear transfer conditions seems to be confirmed by these results. Comparing the selected joint efficiencies tabulated in the previous section with the corresponding test values of Table 2 demonstrates that the pairs of sets ST-SU, SV-SP, and SQ-SR represent the cases of dead, deteriorated, and new joints, respectively. From the data in Table 2 it is noted that the interval of time between the peaks in the displacement plots decreases as the value of in-plane compression increases.

TABLE 1 Maximum Displacements and Joint Efficiency for System 1

	Loaded Node		Unloaded Node		
Test No.	Max. Displ. in./g[a]	Time (msec)	Max. Displ. in./g	Time (msec)	JE[b]
B 1	-5.200 E-5	7.76	-4.680 E-5	7.77	0.900
B 2	-5.000 E-5	6.30	-4.200 E-5	6.38	0.840
B 3	-5.240 E-5	6.99	-3.288 E-5	6.55	0.628
B 4	-4.720 E-5	6.74	-4.040 E-5	6.61	0.856
B 5	-3.864 E-5	6.49	-3.440 E-5	6.51	0.890
B 6	-3.780 E-5	6.63	-4.280 E-5	6.77	1.132
B 7	-3.528 E-5	8.25	-2.924 E-5	8.19	0.829
B 8	-3.024 E-5	7.16	-2.872 E-5	7.05	0.950

[a] g = Acceleration of Gravity
[b] JE = Joint Efficiency

TABLE 2 Maximum Displacements, Time, and Joint Efficiencies for System 2

TEST	LOAD PER ROD (lbs)	LOADED SIDE		UNLOADED SIDE	
		Max. Disp. (in.$*10^{-6}$)	Time (msec)	Max. Disp. (in.$*10^{-6}$)	Time (msec)
ST	2,000	-7.22	6.11	-1.5	7.1
SU	4,000	-6.35	5.4	-2.31	6.16
SV	5,000	-5.99	5.65	-3.17	6.72
SP	6,000	-5.13	5.35	-3.75	5.82
SQ	8,800	-4.02	5.82	-3.78	5.78
SR	9,500	-3.91	5.47	-3.81	5.52

TABLE 3 Frequency Variations on Unloaded Side of Joint

Test	Power Spectrum Peaks' Frequencies, Hz					
ST	244	488	976	1586	/	2562
SU	244	610	976	1586	2440	2562
SV	244	732	1098	1586	2440	2562
SP	244	1098	/	1586	/	2684
SQ	244	1098	/	1708	2440	2684
SR	244	1098	/	1708	2440	/

/ = Peak too small to be significant, or no response.

Also it is noted that the amplitude of the displacements decreases, as does the difference between the values of the displacements on the two sides of the joint at any time, when the in-plane compression forces increase. These behavioral aspects demonstrate that the application of variable in-plane compressive forces to the slabs simulates effectively different aggregate interlock shear transfer conditions across a pavement joint. As the value of this force increases, the two slabs tend to behave more as one. For high values of applied force (for example, sets SQ and SR), the two sides of the interface move almost simultaneously and with displacements of almost the same amplitude.

The information in Table 2 is used to provide further insight into impact load response approaches. It is noted that for joints with good load transfer the peak displacements on both sides of the joint occur at almost the same time (the largest time difference was measured for case SQ 2 at 0.64 msec). However, that time difference increases as the load transfer deteriorates. This indicates that there could be a problem if traditional FWD data are used for the assessment of PCC pavements with poor joints. If one employs the peak displacements from the two sides of the joint, the corresponding joint efficiency ratio is based on events that do not occur at the same time. If, however, the joint efficiency ratio is derived on the basis of readings for the same time, they may not represent the peak displacements on both sides of the joint. This problem does not exist with the present approach, because the entire time history is used to obtain the corresponding power spectra.

The data for all cases were transformed into the frequency domain, and representative results were tabulated for each in-plane force set. Frequency data for each in-plane force set for the unloaded side of the joint are presented in Table 3.

Comparison of Results

The following observations are made on the basis of the results presented in Table 3:

1. The data indicate that the second response frequency is affected by the shear transfer capability of the joint. An increase in the quality of the contact across the joint (i.e., better aggregate interlock) increases the second response frequency, as shown in Figure 2. A further increase in the in-plane force caused the second frequency to reach and remain at 1,098 Hz on the unloaded side. A similar trend was observed from readings on the loaded side.
2. The response of the larger slab (System 1) is very close to the response of the two slabs (System 2) subjected to a large in-plane force. The main difference is in the first frequency, which is a function of the slab's dimensions and dynamic characteristics. This difference, observable for both sides of the joint, indicates that the large slab has a rigid body motion that is different from that of the two smaller slabs.
3. For higher frequencies, no appreciable differences can be related to changes in the quality of the contact between the sides of the joint, and the frequency values on both sides remain almost constant for all the tests.

CONCLUSIONS

An exploratory experimental approach has been presented for nondestructive testing of pavement joints with aggregate interlock load transfer capabilities. The approach is based on the shear vibration response of structural systems under impulsive loads, and on the considerable time separation between shear and flexural behavior modes. Experimental data were acquired on models representing different aggregate interlock shear transfer capabilities across concrete interfaces. Those data were analyzed both in the time domain and in the frequency domain utilizing the Fourier analysis technique.

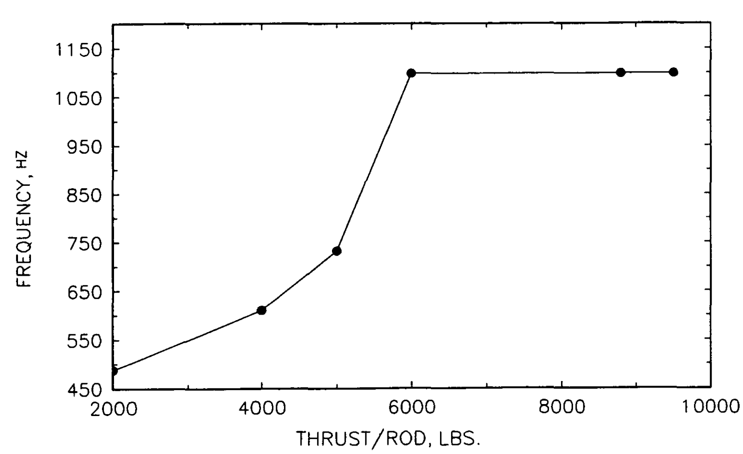

FIGURE 2 Frequency shift and interface contact conditions.

The results obtained confirm the initial expectation that response frequencies in the dynamically loaded structure are closely related to the shear transfer capacity of the interface. The following conclusions are drawn from this study:

1. The model that has been adopted to simulate different aggregate interlock shear transfer conditions is reliable, and it has been confirmed by experimental data.
2. On the basis of the obtained results, frequency variation of modal response is an excellent tool for deciphering the internal aggregate interlock conditions of concrete interfaces. Different vibration frequencies are representative of different values of corresponding joint shear transfer.
3. It is concluded from these results that this approach, after further development, could be employed for the determination of the internal shear transfer capabilities of interfaces in rigid pavement joints.

ACKNOWLEDGMENT

The study reported herein was performed by the University of Minnesota under contract with the Minnesota Department of Transportation. The authors wish to express their deep appreciation to George Cochran and his staff for their cooperation and support.

REFERENCES

1. L. Palmieri and T. Krauthammer. *Vibration Spectroscopy for Rigid Pavement Joint Assessment.* Report MN/RD-91/04. Minnesota Department of Transportation, St. Paul, Aug. 1990.
2. T. Krauthammer and L. Palmieri. Concrete Interface Shear Vibration Spectroscopy. *Journal of Engineering Mechanics*, ASCE, Vol. 117, No. 10, Oct. 1991, pp. 2251–2264.
3. ACI-ASCE Committee 426. The Shear Strength of Reinforced Concrete Members. *ASCE Journal of Structural Division*, Vol. 99, June 1973, pp. 1091–1187.
4. R. Park and T. Paulay. *Reinforced Concrete Structures*. Wiley-Interscience, New York, 1975.
5. G. J. MacGregor. *Reinforced Concrete, Mechanics and Design*, 2nd ed. Prentice Hall, Englewood Cliffs, N.J., 1991.
6. R. I. T. Williams. *Cement-Treated Pavements*. Elsevier Applied Science Publishers, New York, 1986.
7. E. J. Yoder and M. W. Witczak. *Principles of Pavement Design*. Wiley, New York, 1975.
8. H. A. Mattock and N. M. Hawkins. Shear Transfer in Reinforced Concrete—Recent Research. *Journal of Prestressed Concrete Institute*, Vol. 17, No. 2, March–April 1972.
9. P. T. Foxworthy. *Concepts for the Development of a Nondestructive Testing Evaluation System for Rigid Airfield Pavements*. Ph.D. thesis. Department of Civil Engineering, University of Illinois at Urbana-Champaign, June 1985.
10. A. M. Tabataie and E. J. Barenberg. Structural Analysis of Concrete Pavement Systems. *Journal of Transportation Engineering*, ASCE, Vol. 106, No. TE5, Sep. 1980.
11. W. M. Moore, D. I. Hanson, and J. W. Hall. *Transportation Research Circular 189: An Introduction to Nondestructive Structural Evaluation of Pavements*. TRB, National Research Council, Washington, D.C., Jan. 1978.
12. M. S. Hoffman and M. R. Thompson. Comparative Study of Selected Nondestructive Testing Devices. In *Transportation Research Record 852*, TRB, National Research Council, Washington, D.C., 1982.
13. B. Sebaaly, T. G. Davis, and M. S. Mamlouk. Dynamics of Falling Weight Deflectometer. *Journal of Transportation Engineering*, ASCE, Vol. 111, No. 6, Nov. 1985.
14. R. W. Clough and J. Penzien. *Dynamics of Structures*. McGraw-Hill, New York, 1975.
15. A. V. Oppenheim and R. W. Schafer. *Digital Signal Processing*. Prentice-Hall, Englewood Cliffs, N.J., 1975.
16. *Asystant+, Scientific Software for Data Acquisition and Analysis*. Asyst Software Technologies, Inc., Rochester, N.Y., 1987.
17. J. M. Biggs, *Introduction to Structural Dynamics*. McGraw-Hill, New York, 1964.
18. T. Krauthammer, N. Bazeos, and T. J. Holmquist. Modified SDOF Analysis of RC Box-type Structures. *Journal of Structural Engineering*, ASCE, Vol. 112, No. 4, April 1986, pp. 726–744.
19. T. Krauthammer, S. Shahriar, and H. M. Shanaa. Response of RC Elements to Severe Impulsive Loads. *Journal of Structural Engineering*, ASCE, Vol. 116, No. 4, April 1990, pp. 1061–1079.
20. A. Assadi-Lamouki and T. Krauthammer. *Development of Improved Timoshenko Beam and Mindlin Plate Theories for the Analysis of Reinforced Concrete Structures Subjected to Impulsive Loads*. Structural Engineering Report ST-88-02. Department of Civil and Mineral Engineering, University of Minnesota, May 1988.
21. T. Krauthammer and K. L. Western. Joint Shear Transfer Effects on Pavement Behavior. *Journal of Transportation Engineering*, ASCE, Vol. 114, No. 5, Sept. 1988, pp. 505–529.