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Impact Factors for Composite Steel Box Girder Bridges

Coefficients de chocs pour les ponts-mixtes à poutres-maîtresses en caissons

Stosszuschlagsbeiwerte für Verbundbrücken mit Hohlkastenquerschnitt

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SUMMARY

A dynamic measurement program was carried out on a number of steel-concrete composite box-girder bridges with different span lengths and number of spans. The program involved the multi-point measurement of strain and acceleration at selected locations in the bridges due to normal traffic and special test vehicles. The data acquired are used to determine bridge dynamic characteristics such as natural frequencies, modal shapes, impact factors and damping ratios. For the bridges tested, the average impact factor is about 15 to 16 percent. This value is lower than that by the current CSA and AASHTO formula.

RESUME

Une étude de mesures dynamiques fut conduite sur plusieurs ponts en construction mixte acier-béton, à poutres-maîtresses en caisson, avec des travées en nombre et longueurs différents. Cette étude consistait à mesurer la déformation et l'accéleration (à des points spécifiés) de ces ponts, dues à la circulation habituelle et à des véhicules d'essais. Les données recueillies sont utilisées pour déterminer les caractéristiques dynamiques de ces ponts, telles que les fréquences naturelles, les formes modales, les coefficients de chocs et les facteurs d'amortissement. Pour les ponts étudiés, la moyenne des coefficients de majoration pour actions dynamiques est approximativement de 15 à 16 %. Cette valeur se trouve plus petite que celle prescrite par la formule actuelle de CSA et d'AASHTO.

ZUSAMMENFASSUNG

An einer Reihe Verbundbrücken mit unterschiedlichen Spannweiten wurden dynamische Messungen durchgeführt. Das Messprogramm umfasste Dehnungs- und Beschleunigungsmessungen an zahlreichen Punkten ausgewählter Brückenquerschnitte unter normaler Verkehrslast und unter Sonderlasten. Die Messdaten werden für die Bestimmung dynamischer Kennzahlen wie Eigenfrequenz, Eigenschwingungsform, Stosszuschlag und Dämpung verwendet. Für die untersuchten Brücken wurden Stosszuschläge zwischen 15 und 16 Prozent ermittelt. Die Werte sind kleiner als die nach der CSA- und AASHTO-Vorschrift berechneten Werte.

1. INTRODUCTION

Improved materials, new construction techniques and increasingly refined design and analysis procedures for static load on bridges have led to rapid advances in highway bridge engineering in recent years. More economical designs and longer spans have been achieved. In addition, there has been a gradual increase in permissible truck loadings on the roads. However, the vehicle loading and impact specifications in common use in Canadian and American bridge codes (1)(2) have not been modified since 1944. Recent tests in Ontario and in other countries indicate that impact factors considerably different from those used in design can occur in practice.

As a part of an on-going project to develop a data base on the dynamic behaviour of steel-concrete composite box-girder bridges, a preliminary dynamic measurement program was carried out on three such bridges of different sizes (single span, two-span continuous and five-span continuous). This field testing program involved the multi-point measurements of strain and acceleration at selected locations in the bridges, in response to normal traffic and special test vehicles. The results of the data acquired are used to characterize the bridge dynamics by the determination of impact factors, mode shapes and associated natural frequencies as well as giving estimates of the modal damping coefficients.

This paper gives a description of the instrumentation, the tests performed, the data analysis procedures, the results and a summary of the findings and conclusions.

2. DESCRIPTION OF THE BRIDGES

Three steel-concrete composite box-girder bridges were tested between 1978-1980. Pertinent geometrical data of these three structures are shown in Figure 1. All three bridges have solid concrete decks with typical cross-sections as shown. The Kledo River Bridge is a 210'-6" single-span twin box-girder bridge with a girder depth of 8'-1" and an overall deck width of 31'. Each girder of the bridge is supported on two steel bearings at each end. The Kouchibouguac River Bridge is a twin box structure with two 121' spans. The girders have a minimum depth of 3'-3" at the abutments and a maximum depth of 5'-3" at the pier. Two steel bearings per box are used at each support. The Muskwa River Bridge has five spans measuring 181'-3", 300', 300', 180' and 121'3", and an overall deck width of 37'. The structure is a twin steel box composite bridge with a depth varying from 5'-¾" minimum to 12'-6¾" maximum. Similar steel bearings as in the other two bridges are used.

3. INSTRUMENTATION

Data was acquired in the form of bridge strain and acceleration at selected locations, in response to vehicle excitation. The bulk of the data was acquired in response to test vehicles although some data was obtained during passage of normal traffic.

3.1 Strain Gauge

The strain gauges used were AILTCH resistive type weldable gauges. These gauges have a resistance of 120 ohms and a gauge factor of 1.93 $\pm$ 3%. The gauges were mounted in the usual manner, the surface being initially prepared by grinding and buffing to remove the rust and scale, followed by degreasing prior to spot welding. Each gauge was treated as the active arm in a Wheatstone Bridge, and was connected to a dummy foil gauge in a bridge completion unit to complete a half-bridge circuit. This configuration would effectively eliminate any temperature induced strains. Figure 2 shows the locations of the strain gauges.

3.2 Acceleration

The accelerometers used were Bruel and Kjaer 8306 piezeoelectric type accelerometers. They were preconditioned by Borr Brown 3620L amplifiers and had a calibrated output of 1.0 v/m.sec^{-2}. The accelerometers were attached to a heavy block of steel and placed near the edge of the bridge. The locations of the accelerometers were as shown in Figure 2.

3.3 Recording and Display Instruments

All acceleration data and some strain data were recorded onto Ampex ½" #786 magnetic tapes using a RACAL STORE 7D FM tape recorder. A tape speed of 15/16 IPS was employed during all the tests. The STORE 7D has a band width of DC-300 Hz at this tape speed.

Some strain data were also displayed on a Honeywell 8-channel oscillograph. Model 1883(A)-MPD amplifiers, giving a wide range of DC adjustment, were employed with the oscillograph.

A Spectral Dynamics Model 340 FFT analyzer was also used on site. This was useful for getting an idea of the frequency content of the acceleration signals while still in the field.

4. TEST VEHICLES

Two tractor-trailer units were rented locally for each test. Each vehicle was loaded with gravel and had an approximate gross vehicle weight W of about 85 kips (steering axle = 0.1 W, dual axle = 0.45 W and tandem axle = 0.45 W) This gross vehicle weight is roughly 90-95 percent of the standard HS25 vehicle.

5. TESTING PROGRAM

The tests conducted consisted of single, tail-gating and double vehicle tests. In the tail-gating tests, the trailing distance which produces the maximum negative moment at the pier was used. The double vehicle tests involved the two vehicles crossing the bridge side by side. For all tests, the vehicle speed was varied from 5 mph to 65 mph. This was done to study the response of the bridge to various vehicle speeds. The 5 mph runs would produce the crawl (or almost static) response of the bridge. The vehicle speeds and configurations for the tests are summarized in Table 1.

6. DATA ANALYSIS

6.1 Natural Frequencies

To evaluate the natural frequencies of the bridge, it is necessary to determine the frequency contents of the acceleration signals. This is achieved by applying the fast Fourier transform (FFT) procedure to the signals, thus transforming them from the time domain to the frequency domain. This procedure was carried out on the test data using a 1024 point Sande algorithm. The resulting frequency spectra were plotted. The spectral peaks in the frequency domain indicate resonance conditions at the particular frequency values, i.e., the various bending and torsional modes of natural vibration occur at these frequencies. It is generally very difficult to discern which frequency corresponds to a bending mode and which to a torsional mode. One method of separating the modes is to add and subtract the signals from two sensors located at opposing edges of the bridge, and perform a FFT on the resulting signals. By adding the two signals, the majority of the out-of-phase signals will be eliminated. Similarly, by subtracting the two signals, only the out-of-phase signals will remain. Some of the results obtained from this procedure are shown in Table 2. The higher modes are generally of no interest to a bridge engineer, since the most severe damage often takes place at the lower modes of vibration.

6.2 Impact Factor

The impact or dynamic amplication factor, a measure of how much the stresses resulting from the vehicle loads are magnified due to the dynamic nature of the loads, is generally expressed as the ratio of the dynamic component of the strains to its static counterpart at the peak or maximum value. The Canadian CSA code stipulated that the impact factor for a bridge is to be calculated according to the formula $I = 15/(38 + L) \leq 0.30$, where L is the span of the structure in metres ($I = 50/(125 + L) \leq 0.30$ in the American AASHTO code, L in feet). To determine the impact factor from the experimental strain data, it is necessary to extract the static or crawl component from the raw strain data. This was achieved by applying a digital low pass filter (pass band = 0.5 Hz) to the raw data. The results of the calculations are tabulated in Table 3. The maximum impact factors were found to be 27.0 percent for the Kouchibouguac River Bridge and 22.7 percent for the Muskwa River Bridge. These values are slightly higher than those prescribed by the codes. Both maximums were caused by high-speed single vehicle runs. The average value of the impact factors is 11.7 percent for the Kouchibouguac River Bridge and 7.2 percent for the Muskwa River Bridge. These are well below the code values.

6.3 Damping Ratios

Damping is a measure of the ability of the structure to dissipate energy through either mechanical friction or material hysteresis. This quantity is often expressed as a ratio or percentage of the critical damping. Critical damping is the smallest amount of damping for which no oscillation occurs in the free response of the structure. The damping ratio

Table 1 - Typical Testing Configurations

Name of Bridge	Vehicle Config-uration	North Bound						South Bound					
		Speed: (mph)						Speed: (mph)					
		5	20	30	40	50	65	5	20	30	40	50	65
Kledo River Bridge	Single	Yes	Yes	Yes	Yes	Yes	No	Yes	Yes	Yes	Yes	Yes	No
	Multiple Vehicle	Yes	Yes	Yes	Yes	Yes	No	Yes	Yes	Yes	Yes	Yes	No
	Tail-gating	No	No	No	No	No	No	No	No	No	No	No	No
Kouchibouguac River Bridge	Single	Yes	Yes	Yes	Yes	Yes	No	Yes	Yes	Yes	Yes	Yes	No
	Multiple Vehicle	Yes	Yes	Yes	Yes	Yes	No	Yes	Yes	Yes	Yes	Yes	No
	Tail-gating	Yes	Yes	Yes	Yes	Yes	No	Yes	Yes	Yes	Yes	Yes	No
Muskwa River Bridge	Single	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
	Multiple Vehicle	Yes	Yes	No	No	Yes	No	Yes	Yes	No	No	Yes	No
	Tail-gating	Yes	Yes	No	No	Yes	No	Yes	Yes	No	No	Yes	No

Table 2 - Natural Frequencies (Hz) and Corresponding Modal Shapes

Mode Number	Kledo River Bridge		Kouchibouguac River Bridge		Muskwa River Bridge	
1	1.47	B	1.8	B	0.9	B
2	1.76	T	2.9	B	1.45	B
3	3.93	T	4.2	T	2.2	B
4	4.74	T	4.5	T	2.4	B
5	5.11	B	8.2	B	2.9	B

B = Bending Mode
T = Torsional Mode

Table 3 - Mean Value of Impact Factors

3.1 Kouchibouguac River Bridge

	Gauge Number	North Bound Speed: (mph)					South Bound Speed: (mph)					Remarks
		20	30	40	50	65	20	30	40	50	65	
Single Vehicle	S2-5	4.6	*	9.1	14.7	-	*	7.5	15.5	27.0	-	* data discarded
	S2-7	19.3	*	11.4	16.0	-	23.3	8.8	18.0	26.1	-	- no measurement taken
	S2-8	9.9	9.7	9.8	8.2	-	4.3	-	9.5	6.3	-	
Multi Vehicle	S2-5	7.3	7.5	10.8	15.7	-	14.2	12.8	12.4	8.9	-	
	S2-7	13.7	8.8	11.4	20.0	-	21.3	8.6	9.5	14.9	-	
	S2-8	3.3	7.2	5.9	7.1	-	5.7	12.3	7.5	4.3	-	
Tail-gating	S2-5	13.9	5.8	17.3	4.3	-	15.9	15.3	19.5	7.7	-	
	S2-7	13.4	14.2	22.0	6.6	-	16.8	17.4	14.0	13.4	-	
	S2-8	5.7	9.8	5.3	6.5	-	9.2	15.1	8.5	6.5	-	

3.2 Muskwa River Bridge

	Gauge Number	North Bound Speed: (mph)					South Bound Speed: (mph)					Remarks
		20	30	40	50	65	20	30	40	50	65	
Single Vehicle	S3-1	*	*	*	*	*	*	*	*	*	*	* data discarded
	S3-2	7.3	4.7	9.1	9.8	15.1	3.3	3.8	9.1	11.9	12.2	- no measurement taken
	S3-3	4.5	4.4	7.6	7.5	12.5	3.9	5.6	8.3	8.0	9.3	
	S3-4	5.8	5.0	6.4	6.7	11.7	7.1	11.1	8.6	10.0	7.7	
	S3-5	3.4	4.7	5.8	3.8	4.4	5.0	3.5	7.1	3.0	3.7	
	S3-6	6.3	4.9	4.2	4.3	3.6	3.0	3.1	4.5	4.9	4.2	
	S3-7	4.4	3.0	5.4	7.2	11.8	5.0	8.2	19.4	22.7	-	
Multiple Vehicle	S3-1	9.7	9.3	6.2	12.5	-	10.1	-	-	11.0	-	
	S3-2	5.5	-	3.7	7.1	-	5.9	-	-	10.5	-	
	S3-3	5.9	5.9	8.6	8.6	-	7.2	-	-	7.3	-	
	S3-4	6.3	5.6	5.5	5.7	-	7.4	-	-	8.2	-	
	S3-5	3.3	3.3	2.5	3.3	-	4.0	-	-	6.0	-	
	S3-6	2.5	4.2	2.5	3.4	-	3.6	-	-	4.9	-	
	S3-7	5.6	10.9	6.7	12.0	-	7.3	-	-	6.7	-	
Tail-gating	S3-1	10.5	-	-	15.6	-	7.1	-	-	15.1	-	
	S3-2	8.5	-	-	11.2	-	5.0	-	-	11.5	-	
	S3-3	8.1	-	-	13.7	-	7.2	-	-	10.6	-	
	S3-4	7.2	-	-	8.3	-	9.8	-	-	10.7	-	
	S3-5	4.1	-	-	6.3	-	5.2	-	-	6.2	-	
	S3-6	4.8	-	-	5.0	-	5.0	-	-	6.8	-	
	S3-7	6.2	-	-	9.4	-	11.0	-	-	15.2	-	

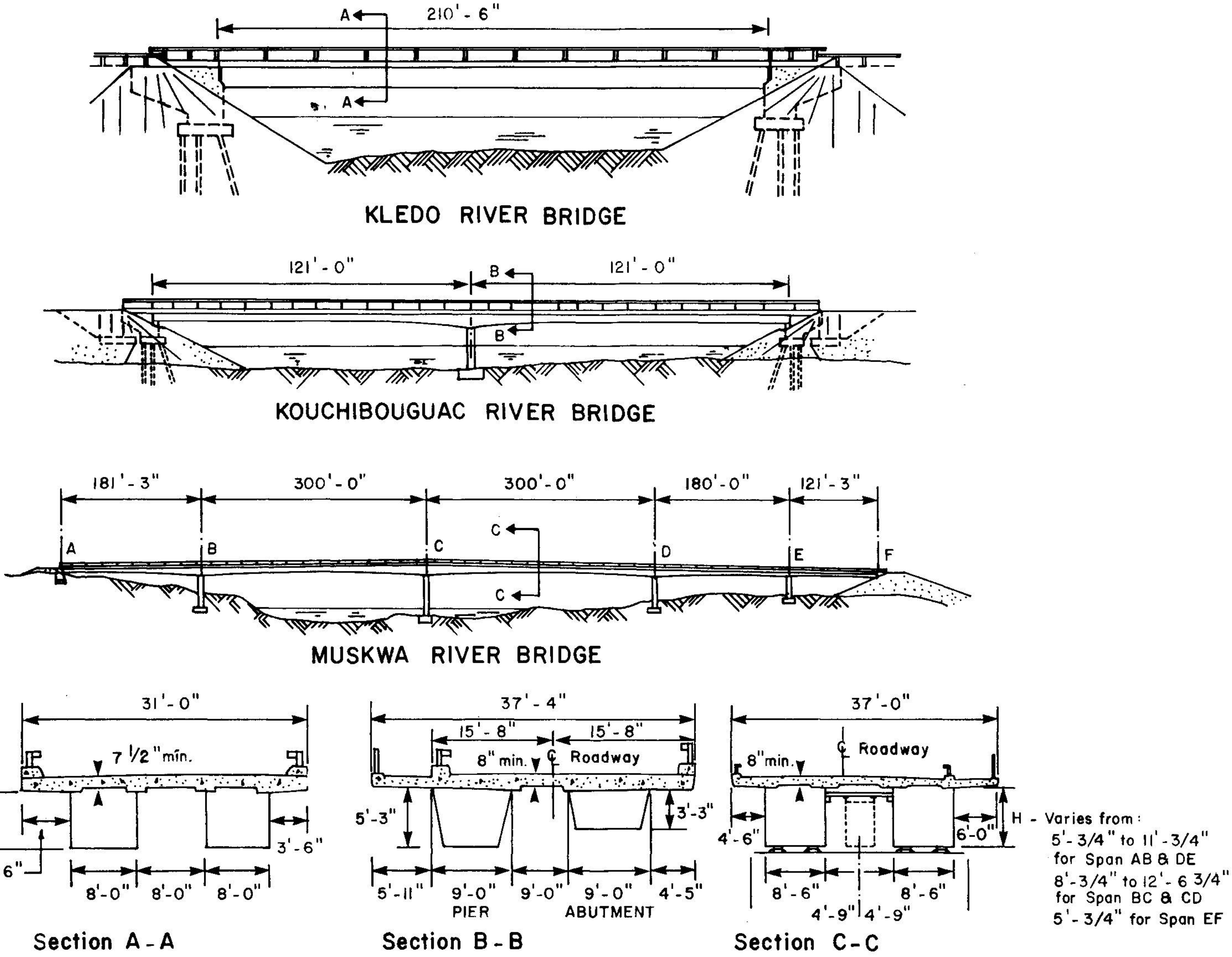

Figure 1 Elevations and Cross sections (1 ft. = 0.3048m, 1 in. = 25.4mm)

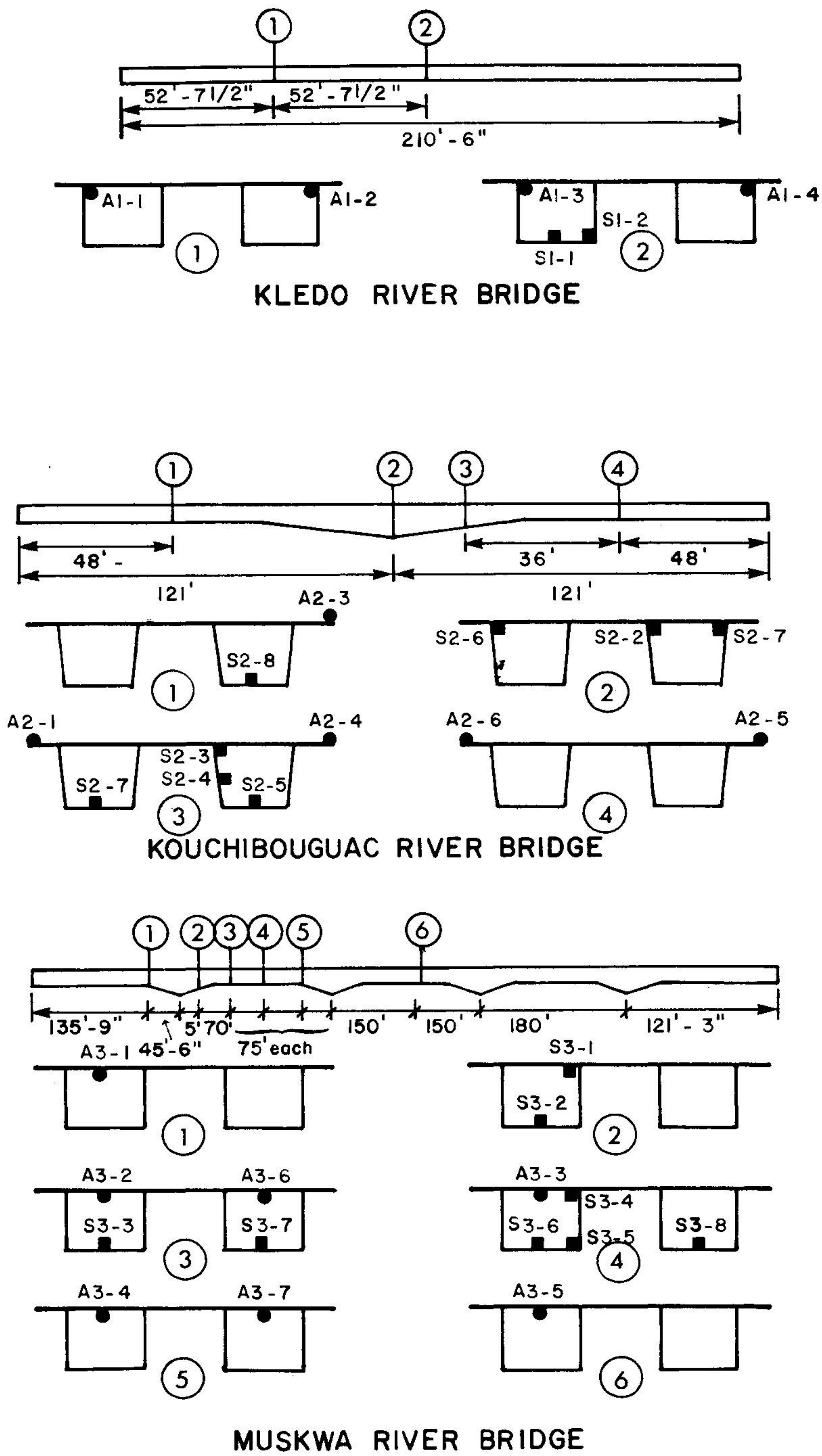

Figure 2 Strain Gange and Accelerometer Layout

can be determined by least square curve fitting an exponential decay curve through the first 10 to 20 cycles of the filtered acceleration signal. Applying this procedure to a number of runs on the Kouchibouguac River Bridge it was found that the damping ratios are 0.5 percent at 1.8 Hz, 0.4 percent at 2.9 Hz, and 0.6 percent at 4.2 Hz. These values are quite low compared to typical values of 1.0 to 2.0 percent found in most box-girder bridges. This implies that the structure is very lightly damped, i.e., the time required for the vibration to die down is comparatively long. The damping ratios for the Muskwa River Bridge are 1.4 percent at 0.9 Hz, 2.0 percent at 1.45 Hz, and 2.4 percent at 2.2 Hz. These values are typical of long span composite box-girder bridges.

7. SUMMARIES AND CONCLUSIONS

Fundamental frequencies of the steel-concrete composite box-girder bridges tested all lie between 1-2 Hz and are associated with a bending or flexural mode.

Maximum impact factors are found slightly higher than those prescribed by the codes. However, the average value of the impact factors is generally below the code value. Additionally, it should be recognized that these maximum impact factors are caused by single vehicle crossings which in general produce lower bending moments.

Impact factors corresponding to multiple vehicle configurations generally produce lower impact factors than a single vehicle configuration.

Mid span locations generally exhibit lower impact factors while quarter span and pier locations tend to yield higher values.

There is a general correspondence between higher impact values and higher vehicle speed.

8. ACKNOWLEDGEMENTS

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