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BRIDGE SYSTEM IDENTIFICATION FROM MODAL INFORMATION

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Dynamic loading and testing of bridges in Ontario[1]

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The Ontario Highway Bridge Design Code (OHBDC) contains provisions on dynamic load and vibration that are substantially different from other codes. Dynamic testing of 27 bridges of various configurations, of steel, timber, and concrete construction, and with spans from 5 to 122 m was therefore undertaken to obtain comprehensive data to support OHBDC provisions. Standardized instrumentation, data acquisition, and test and data processing procedures were used for all bridge tests. Data was gathered from passing trucks, and scheduled runs by test vehicles of various weights. Accelerometer responses were used to determine bridge vibration modes, and dynamic amplifications were obtained from displacement or strain measurements. The form of the provisions adopted for dynamic load and vibration was confirmed by the test results, subject to minor adjustment of values. Observations on the distribution of dynamic load, and its relationship to span length and vehicle weight, may provide a basis for future refinement of the dynamic load provisions. If the stiffness of curbs and barrier walls is not included in deflection calculations, bridges designed by deflection could be penalized.

Key words: bridges, vibration, bridge testing, bridge design codes.

Le code Ontarien traitant du calcul des ponts routes (OHBDC) contient des clauses concernant les charges dynamiques et les vibrations qui sont substentiellement différentes de celles des autres normes. Afin d'établir ces clauses à partir de données sûres, des essais dynamiques ont dès lors été effectués sur 27 ponts de types différents, construits en acier, bois et béton et de portées variant de 5 jusqu'à 122 m. Tous ces essais furent réalisés suivant la même procédure, en utilisant la même instrumentation et permirent d'obtenir les mêmes types de données. Des mesures furent effectuées lors du passage de camions ainsi que lors du passage de véhicules d'essais de poids différents. Les résultats des accéléromètres ont permis de déterminer les différents modes de vibration du pont, tandis que les amplifications dynamiques furent obtenues à partir des mesures de déplacements et de déformations unitaires. Ces essais ont démontré le bien fondé des clauses traitant des charges dynamiques et des vibrations, seuls de légers changements aux valeurs numériques ont été nécessaires. Les observations sur la répartition des charges dynamiques en rapport avec la portée et le poids de véhicules peuvent être utilisées afin d'améliorer les clauses concernant les charges dynamiques. Les cas où les raideurs des troittoirs et garde-fou ne sont pas considérées pénalisent les ponts calculés à partir des déplacements.

Mots clés: ponts, vibration, essai de pont, normes de calcul de ponts.

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1. Introduction

The first edition of the Ontario Highway Bridge Design Code (OHBDC) contained significant changes from other North American codes in provisions regarding dynamic loading and dynamic response of bridges, necessitated in part by the limit states philosophy of the new code (MTC 1979; Billing and Green 1984). The principal provisions were

(1) A dynamic load allowance of 0.4 for deck slabs, whose design would be governed by a single axle unit and where dynamic loading would be due to axle hop.

(2) A dynamic load allowance of 0.35 for other slabs, and beams having spans up to 12 m, whose design would be governed by a dual axle unit or part of a truck and where dynamic loading would be due to axle unit dynamics.

(3) A dynamic load allowance dependent upon the unloaded first flexural frequency of the structure (Fig. 1) for other main longitudinal members, whose design would be governed by a truck or lane loading, and where dynamic loading would be due to response of the structure in its modes of vibration.

(4) A criterion in terms of superstructure deflection to limit vibration that might be perceived by pedestrians on the bridge, dependent upon both anticipated pedestrian usage of the bridge and the first flexural frequency of the bridge (Fig. 2).

Except for timber bridges, there were no generalized limitations such as deflection-to-span or span-to-depth ratios. The usual intents of such criteria are to limit vibration, control stresses in secondary members, and guard against fatigue failure. These were all covered explicitly in OHBDC.

These provisions attempted to represent as well as possible, in a simple manner, the various mechanisms

[1]This paper was presented at the International Conference on Short and Medium Span Bridges, Toronto, Ontario, 1982.

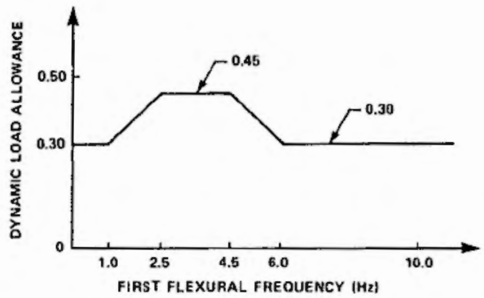

FIG. 1. Dynamic load allowance.

of vehicle and bridge interaction. Analysis of a simple span traversed by a sinusoidally varying force or a sprung mass shows much greater response of the span when there is a match between its frequency and the frequency of the force or mass (Fryba 1972). The curve shown in Fig. 1 represents the likelihood of greater response of bridges having vibration modes with frequencies in the 2–4 Hz range, as this is the frequency range of truck body bounce on tires and suspension. Earlier bridge tests by the Ontario Ministry of Transportation and Communications (MTC) observed this phenomenon (Campbell *et al.* 1971) as have also comprehensive tests conducted in Switzerland (Cantieni 1983).

The specified dynamic load allowances were based on existing data on dynamic amplification (Wright and Green 1964; Campbell *et al.* 1971). Certain assumptions were made regarding the variability of dynamic loading and adjustments were made so that the same load factor was used for live load and dynamic load. However, much of the existing data was based on unknown assumptions and there was very little of a suitable statistical nature. Further, early use of OHBDC resulted in difficulties in meeting the deflection criterion for some bridges, so the validity of the criterion was questioned. It was therefore decided to perform dynamic tests on a representative selection of bridges to generate the data required to develop fully the ideas in OHBDC.

2. The bridges

The bridges for test were selected by inspection of structures on or over the King's Highway in a convenient region of Southern Ontario. Choice was limited to locations without impediment to instrumentation or conduct of the test, and to bridges of typical construction.

Bridges were selected at 22 locations, 5 of which were twin bridges. They included 14 of steel with spans from 22 to 122 m, 10 of concrete with spans from 16 to 41 m, and 3 of timber construction, all with spans of about 5 m. Characteristics of these bridges are summarized in Table 1. The approaches, expansion joints, and decks of all these bridges were in good to excellent condition. There were a few cases of moderate settlement of approach fill, or small discrete bumps, but at no site was there significant breakup or repair of the riding surface that could be described as rough. Approach and deck profiles were not measured.

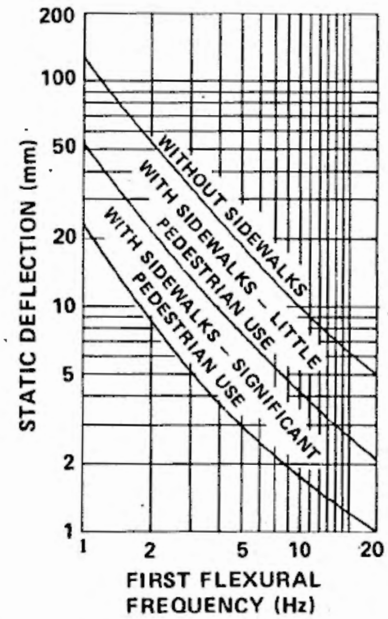

FIG. 2. Deflection limits for serviceability.

3. Test procedure

Testing of 27 bridges in diverse locations in only 3 months, while sharing staff and equipment with concurrent test programs being conducted by MTC, was a demanding logistical problem. It was successfully achieved through the availability of experienced technical staff, equipment, and procedures proven through many years of bridge testing by MTC (Bakht and Csagoly 1979), and good weather. A standard test plan, including instrumentation layout, test procedure, and data processing procedure, was devised so that each test could be conducted in the same manner (Billing 1982).

3.1 Instrumentation and data acquisition

Six Bruel and Kjaer Model 8106 seismic accelerometers were placed on the curb or beside the barrier walls of a bridge so that its lowest frequency flexural and torsional vibration modes could be determined. A pair of pressure switches, each activated by a sealed hose nailed to the roadway 12.192 m apart, was used in each traffic lane to determine vehicle presence and speed. Up to 16 data channels were available for displacement transducers or electrical resistance

TABLE 1. Test bridges

No.	Name	Construction	Spans (m)	Skew
1	Shawanaga River Hwy. 69	4 steel plate girders	1 simple (36.576)	None
2	Little Rouge Creek Hwy. 7	6 AASHTO II girders	3 continuous (15.240, 18.288, 15.240)	15°
3	Black River South Hwy. 48	Timber	5 continuous (4.039, 4.877 × 3, 4.039)	None
4	Jock River Hwy. 16	5 steel plate girders	1 simple (36.576)	None
5	South Nation River Hwy. 417	2 steel box girders	3 continuous (53.035, 66.446, 53.035)	12°
6	Innisville Hwy. 7	5 rolled steel girders	5 simple (22.555 each)	None
7	Black River Hwy. 7	4 reinforced concrete girders	2 simple (15.850 each)	None
8	Millhaven Creek (EBL) Hwy. 401	Rigid frame	1 rigid (16.764)	15°
9	Millhaven Creek (WBL) Hwy. 401	Rigid frame	1 rigid (16.764)	15°
10	Palace Road (EBL) Hwy. 401	8 rolled steel girders	1 simple (16.801)	46°27′
11	Palace Road (WBL) Hwy. 401	8 rolled steel girders	1 simple (16.801)	46°27′
12	Eagleson's Road over Hwy. 401	5 prestressed concrete girders	1 simple (41.377)	39°42′
13	Baxter Creek Hwy. 28	6 AASHTO II girders	1 simple (16.459)	None
14	Cavanville Creek Hwy. 28	Prestressed concrete slab	3 continuous (8.534, 12.802, 8.534)	None
15	Pefferlaw Brook Hwy. 48	8 AASHTO II girders	3 continuous (12.192, 21.336, 12.192)	10°
16	South Muskoka R. (NBL) Hwy. 11	Deck truss	4 continuous (7.620, 20.523, 30.785, 7.696)	None
17	South Muskoka R. (SBL) Hwy. 11	3 steel box girders with stay legs	3 continuous (25.908, 36.576, 25.908)	None
18	Gull Lake (NBL) Hwy. 11	4 steel plate girders	2 continuous (48.768, 48.768)	None
19	Gull Lake (SBL) Hwy. 11	4 steel plate girders	2 continuous (48.768, 48.768)	None
20	North Muskoka R. (NBL) Hwy. 11	5 steel plate girders	1 simple (45.720)	None
21	North Muskoka R. (SBL) Hwy. 11	Through truss	1 simple (45.720)	None
22	Haultain's Hwy. 28	Timber	10 continuous (4.039, 4.877 × 8, 4.039)	None
23	Townline Road over Hwy. 402	4 precast concrete box girders	2 continuous (30.480, 30.480)	None
24	Bostwick Road over Hwy. 402	Prestressed concrete voided slab	2 continuous (36.576, 38.100)	20°
25	Black River North Hwy. 48	Timber	5 continuous (4.039, 4.877 × 3, 4.039)	None
26	Mississippi River Hwy. 7	5 steel plate girders	5 continuous (21.336, 27.432 × 3, 21.336)	None
27	Madawaska River Hwy. 17	4 steel plate girders	3 continuous (68.58, 121.92, 68.58)	None

NOTE: EBL = eastbound lanes; WBL = westbound lanes; NBL = northbound lanes; SBL = southbound lanes.

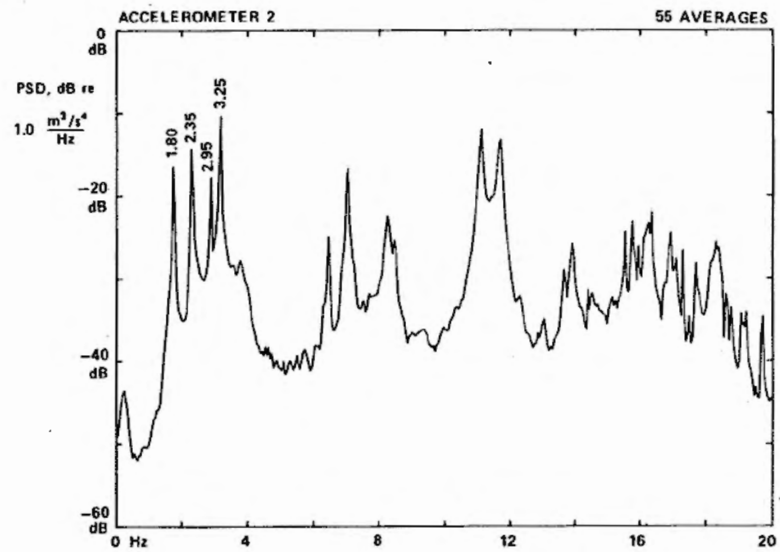

FIG. 3. Power spectra, Gull Lake (SBL).

strain gauges to measure bridge responses. The BLH displacement transducer used was based on a strain-gauged foil loop. It had a range of about 32 mm, and was used for most bridges. It was generally bolted to the underside of a slab or clamped to a girder, and a taut cable was weighted to the ground or riverbed below. Strain gauges were used for steel bridges where the waterway beneath prevented use of displacement transducers.

The data acquisition system was mounted in the MTC Mobile Laboratory. It consisted of three Metraplex 300 units, each of which multiplexed eight data channels onto one track of a Honeywell 5600C 7 track FM instrumentation tape recorder. IRIG B Time Code was recorded on a fourth track. A control panel provided calibration and switching facilities, and was used to demultiplex one track of the recorded signal onto an oscillograph for the test engineer to monitor the test in progress.

3.2 Test vehicles

Four test vehicles owned by MTC were used: TV1 and TV2 were five-axle tractor trailers having gross weights of 391 and 414 kN; TV3 was an eight-axle combination having a gross weight of about 580 kN; and TV4 was the three-axle Inspector 50 service vehicle having a gross weight of about 241 kN. These vehicles were all loaded close to their legal limits, and are representative of heavy commercial vehicles operating in Ontario.

3.3 Test procedures

Instrumentation and the data acquisition system were installed and checked carefully and any problems were resolved by an orderly and disciplined procedure. A calibration of the data acquisition system was recorded on tape.

Each test vehicle made a run driving at 16 km/h in each traffic lane, and close beside the right-hand barrier wall or curb. These low speed runs resulted in bridge responses to vehicles of known weight, permitting calibration of the bridge as a scale for estimation of the weight of passing trucks.

Scheduled runs were then made by the test vehicles driving in each traffic lane at speeds of 48 km/h, the maximum legal speed, and the mean of these. Test runs were also made at the same speeds with test vehicles following each other closely, and on some one-way bridges, with the test vehicles side by side or in echelon. Runs were only made in normal traffic lanes, with traffic controlled by a police cruiser and MTC flagmen to ensure no other vehicles were on the bridge. Positive traffic control ensured no hazard to the travelling public, and kept delays to only 1 or 2 min.

Data was generally recorded from 100 or more individual runs by test vehicles, and also from trucks that passed during the test. A run log of time, truck type, lane of travel, speed, test run number, and other notes was recorded manually. The response of the bridge to passage of trucks was assessed subjectively by technicians standing on the bridge.

3.4 Data processing

The FM tapes recorded during the test were returned to the MTC Research Laboratory for processing. A Nicolet 660A dual channel Fourier analyser was used to determine frequencies, shapes, and damping ratios of the bridge vibration modes (Billing 1982). Selected

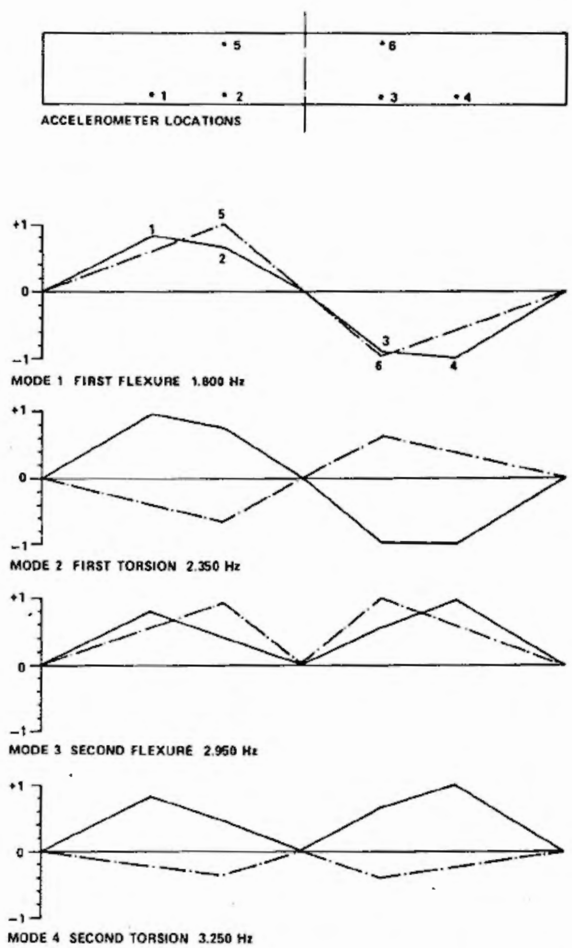

FIG. 4. Bridge vibration mode shapes, Gull Lake (SBL).

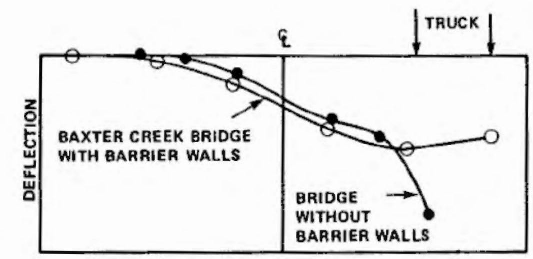

FIG. 5. Stiffening effect of barrier walls.

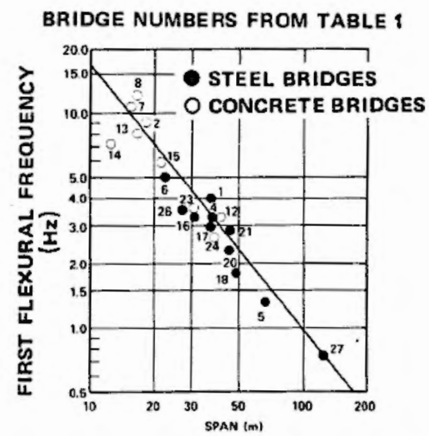

FIG. 6. First flexural frequency vs. span.

vehicle passages of interest were digitized and stored on an IBM-compatible tape, for processing on the IBM 3033 computer system at MTC (Billing 1982).

4. Dynamic characteristics

Frequencies and shapes of the vibration modes of each bridge were determined using the Fourier analyzer (Billing 1982). The number of modes found depended upon the particular characteristics of a bridge. For the longer-span bridges, especially the continuous ones, between 6 and 12 vibration modes of longitudinal flexure, torsion, and transverse flexure could usually be identified with certainty. In contrast, none of the three timber bridges tested appeared to have any vibration modes. Modes of the two bridges consisting of several identical simple spans were difficult to distinguish because each span apparently has a slightly different frequency. Vibration of one span also appeared to be transmitted to other spans, presumably through some or all of the expansion joints, bearings, and piers, and the ground. Damping ratios of vibration modes were determined from the logarithmic decrement of typical bridge responses (Billing 1982).

An example of power spectral density from the Gull Lake (SBL) bridge is shown in Fig. 3. This bridge has two continuous spans, each 48.768 m long. A sharp peak is generally due to a vibration mode, and the low peak below 1 Hz is due to static deflection of the bridge under the moving trucks. Shapes of the first four modes of this bridge are shown in Fig. 4.

A summary of modal frequencies, damping ratios, and mode type is presented for all bridges tested in Table 2. No calculations of frequencies were made for comparison with the test results. Detailed dynamic analyses would undoubtedly provide valuable insight into the contribution of various components to the dynamic characteristics of bridges. It was, however, evident from transverse static response distributions that barrier walls and curbs, whether continuous or not, do provide substantial stiffness under the service loading of a single truck. This stiffness is not considered in design. An example is shown in Fig. 5 where mid-span deflections of the girders of Baxter Creek bridge, with discontinuous New Jersey style barrier walls, are compared with deflections of the girders of a similar three-span continuous bridge without

TABLE 2. Bridge natural frequencies and damping ratios

	Mode 1			Mode 2			Mode 3			Mode 4			Mode 5		
No.	f	γ	MT	f	γ	MT	f	γ	MT	f	γ	MT	f	γ	MT
1	4.00	—	1X	5.13	—	1T									
2	8.88	0.9	1X	10.06	—	1T	14.25	1.4	3X	18.38	0.8	3T			
4	3.13	2.3	1X	3.94	1.7	1T									
5	1.31	0.8	1X	2.00	1.0	2X	2.44	1.7	3X						
6	5.00	—	1X	6.25	—	1T									
7	10.63	3.8	1X	14.06	1.5	1T	15.5	1.6	2T						
8	12.00	1.9	1X	15.44	1.4	1T									
11	10.38	2.7	?												
12	3.13	1.2	1X	3.94	0.7	1T	7.25	—	1Y	11.25	1.4	2X	11.63	1.1	2T
13	8.06	3.1	1X	9.63	1.1	1T	11.38	1.7	1Y	16.25	3.3	2Y			
14	7.13	0.8	1X	9.19	0.9	1T	12.94	1.1	3X	14.06	1.1	1Y			
15	5.88	2.0	1X	6.31	4.0	1T									
16	3.31	0.5	1X	5.31	0.5	1T									
17	2.94	0.4	1X	4.56	0.9	2X	4.81	0.6	3X						
19	1.80	0.4	1X	2.35	0.4	1T	2.95	0.3	2X	3.25	0.3	2T	6.50	—	3X
20	2.31	0.5	1X	2.81	1.2	1T	5.44	—	1Y	5.81	—	2Y	7.06	1.4	3Y
21	2.88	0.7	1X	5.06	—	1T									
23	3.63	3.7	1X	4.94	—	2X	5.69	—	1T	9.38	—	2T	12.94	1.8	3X
24	1.69	3.1	P	2.63	1.4	1X	3.31	0.9	2X	6.63	1.0	2T	7.38	1.7	1T
26	3.44	1.6	1X	4.44	1.3	2X	5.38	—	2T	5.81	1.0	3X	6.81	0.8	3T
27	0.75	0.7	1X	1.47	1.2	2X	2.03	0.9	3X	2.31	—	3T			

NOTE: No. = bridge number, Table 1; f = mode frequency, Hz; γ = damping ratio, % critical; MT = mode type: nX = nth longitudinal flexural, nT = nth torsion, nY = nth transverse flexure, P = pier.

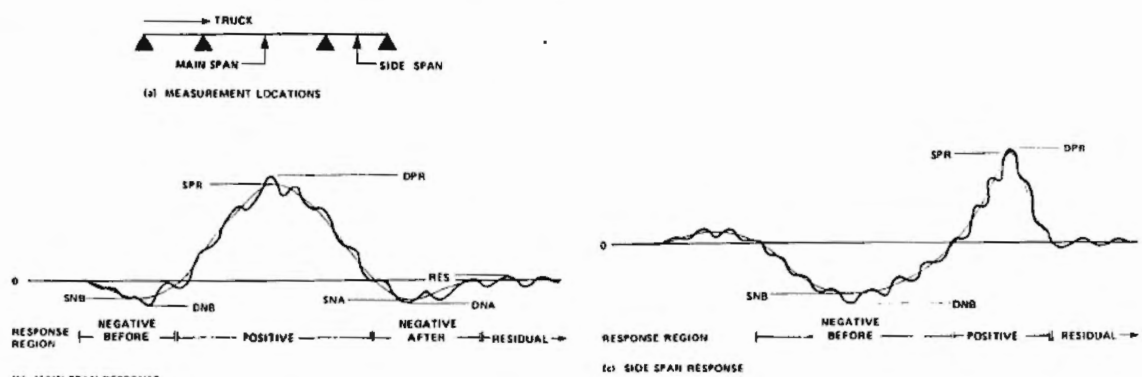

FIG. 7. Schematic of bridge responses.

barrier walls previously tested by MTC (Holowka and Csagoly 1980).

The first flexural frequency is shown in Fig. 6 against longest span of the bridge. While there is a clear trend, the small number of bridges and diversity of their construction make it unreasonable to extract any simple relationship between frequency and span that would be sufficiently accurate for use with OHBDC.

Damping ratios of first flexural modes of steel bridges are generally 0.4–0.7% of critical. Damping ratios of concrete bridges cover a wide range, 0.8–3.8% of critical. These values are in accordance with other damping measurements (Cantieni 1983; Tilly 1977).

5. Dynamic response

5.1 Data analysis

The dynamic response was obtained by processing the digital data of each truck pass on an IBM 3033 computer system. For each run, each data channel was independently zeroed, then filtered with a low-pass digital filter to remove the vibratory components of bridge response to provide an equivalent static response. Three response regions were defined, illustrated by the typical

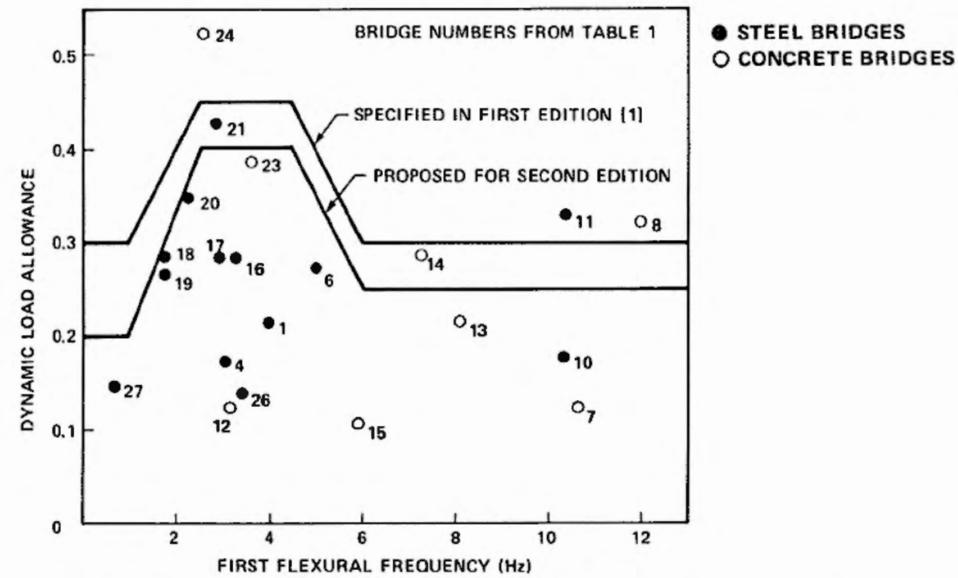

FIG. 8. Comparison with code of calibrated dynamic load allowance from tests.

displacement or strain responses shown in Fig. 7 for a three-span continuous bridge. The positive region is when the truck is on the span containing the instrument. The negative region is when the truck is on a span adjacent to that containing the instrument. The residual region is when the bridge is in free vibration after the truck has left it. A simple span has no negative regions. The response regions were determined by scanning the static response of a designated data channel for each lane and span to determine the period during which the truck was on that span. The quantities shown in Fig. 7 were then determined by scanning each data channel during the appropriate periods.

Dynamic amplifications were computed for the three response regions from the data channel having greatest static response SPX, any other data channel having static response within 10% of SPX, and any data channel having total response greater than the total response of that having greatest static response. Dynamic amplifications (DA) were computed for each qualifying data channel as given in [1].

$$\text{DA} = (\text{DPR} - \text{SPR})/\text{SPX} \quad \text{for the positive region}$$

$$[1] \quad \text{DA} = \text{Max}\,(\text{DNB} - \text{SNB}, \text{DNA} - \text{SNA})/\text{SPX} \quad \text{for the negative region}$$

$$\text{DA} = \text{RES}/\text{SPX} \quad \text{for the residual region}$$

where the variables are as defined in Fig. 7.

The dynamic amplifications for all qualifying data channels were relative to the largest positive region static response. Dynamic amplifications in the three regions could therefore be directly compared. A dynamic amplification is thus not overestimated, as it would be if computed from the static and total responses of a noncritical data channel. Statistics of dynamic amplification for single trucks were computed as functions of truck type, lane of travel, weight, and speed. This data generally included at least 100 trucks weighing between 150 and 600 kN, and was considered representative of the service loading of the bridge.

5.2 Overall statistics of dynamic amplification

The overall statistics of dynamic amplification are presented in Table 3. This data includes, for each bridge, all single truck runs by test vehicles and traffic at all speeds in any lane. It also includes runs by test vehicles following each other closely. The mean dynamic amplifications are seen to be relatively modest, even though for some tests dynamic amplifications greater than 0.5 were observed. The coefficients of variation of dynamic amplification are large, varying between 0.56 and 1.11, with a mean of about 0.82. This is considerably greater than the value of 0.45 assumed in calibration of OHBDC (Lind and Nowak 1978). The values in Table 3 may be compared with OHBDC provisions by means of the calibration process.

$$[2] \quad I = \bar{I}(1 + SV\beta)/\alpha$$

where I = specified dynamic load allowance, $\bar{I}$ = mean observed dynamic amplification, S = sensitivity factor = 0.57 (Lind and Nowak 1978), V = coefficient of variation of dynamic amplification, β = safety index = 3.5 (Lind and Nowak 1978), and α = load factor specified for dynamic load = 1.4 (MTC 1979).

TABLE 3. Overall statistics of dynamic amplification

	Response region						
	Positive		Negative		Residual		
No.	Mean	CoV	Mean	CoV	Mean	CoV	Loc
1	0.129	0.67			0.123	0.77	
4	0.069	0.74			0.097	0.59	
6	0.136	0.90			0.107	0.78	
7	0.057	1.00			0.060	0.73	
8	0.110	1.11			0.104	0.66	
9	0.305	0.91			0.229	0.95	
10	0.093	0.84			0.065	1.04	
11	0.156	0.98			0.106	0.78	
12	0.077	0.65			0.104	0.60	
13	0.098	1.04			0.060	1.23	
14	0.150	0.85	0.105	1.18	0.080	1.17	a
	0.119	0.69	0.033	0.88	0.092	0.67	b
15	0.068	0.61	0.006	1.00	0.012	0.49	a
	0.031	0.72	0.003	0.76	0.018	0.55	b
16	0.161	0.72	0.134	0.79	0.184	0.51	c
	0.205	0.77	0.104	0.98	0.210	0.54	d
17	0.164	0.70	0.123	0.42	0.125	0.46	a
	0.141	0.72	0.100	0.57	0.126	0.47	b
18	0.191	0.55	0.192	0.56	0.360	0.48	
19	0.174	0.56	0.171	0.48	0.247	0.41	c
	0.112	0.60	0.084	0.66	0.147	0.49	d
20	0.194	0.76			0.195	0.62	
21	0.210	0.93			0.190	0.82	c
	0.167	0.82					e
23	0.177	1.03	0.137	0.85	0.078	0.48	
24	0.236	1.05	0.204	0.78	0.147	0.63	
26	0.079	0.73	0.097	0.63	0.120	0.70	a
	0.062	0.83	0.041	0.85	0.069	0.63	b
27	0.090	0.63	0.092	0.52	0.108	0.50	
	0.099	0.67	0.061	0.66	0.097	0.59	b
	0.084	0.59	0.075	0.55	0.131	0.64	d

NOTE: No. = bridge number, Table 1; Loc = location: a = main span, b = side span, c = mid span, d = support, e = floor beam; CoV = coefficient of variation = standard deviation/mean.

The positive region dynamic responses were computed by [2] and are shown in Fig. 8 in comparison with the OHBDC provision. The form for dynamic load allowance adopted for the Code thus appears justified. The values proposed for the second edition were based upon a reasonable upper bound for the test data. The exceptional response of Bostwick Road Overpass (Point No. 24) was due to settlement at the northbound approach while the southbound approach was smooth. The mean observed dynamic amplification of 0.236 is in fact composed of a northbound mean of 0.456 and a southbound mean of 0.059, which resulted in a high coefficient of variation and a high dynamic load allowance.

5.3 *Effects of truck suspension and weight*

Test vehicles TV1 and TV2 were similar in overall dimensions and weight, but TV2 has an air suspension while TV1 has leaf springs. The mean dynamic amplification for all runs on all bridges by TV2 was about 60% of that for TV1. This is presumably because the air spring and parallel shock absorber provide damping under all conditions, whereas the leaf spring only absorbs significant energy for large displacement or rate of loading (AASHTO 1962; Winkler *et al.* 1980).

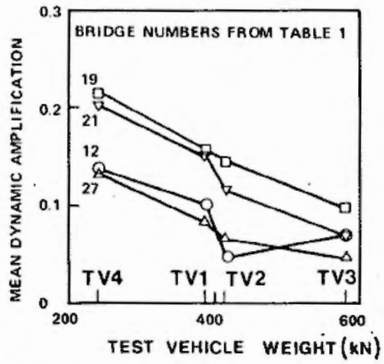

FIG. 9. Mean dynamic amplification vs. test vehicle weight.

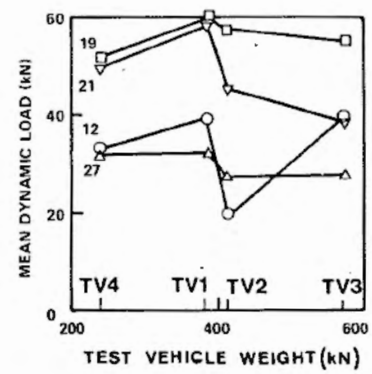

FIG. 10. Mean dynamic load vs. test vehicle weight.

The mean dynamic amplification for all runs by each test vehicle generally decreased with increase in weight for the truck for bridges with spans greater than about 30 m. Some examples are shown in Fig. 9. This is presumably because the additional trailers and axles required for increased gross weight are not all in phase, which moderates the dynamic effect in comparison with an almost linear increase in static effect with gross weight. Defining the product of truck weight and dynamic amplification to be dynamic load, Fig. 10 shows the dynamic load corresponding to the data of Fig. 9. The dynamic load is seen to remain sensibly constant, except in one case where TV2 had a small response. A similar result has been found in other unpublished tests by MTC. The specified dynamic load allowance in OHBDC was based on a calibration of the

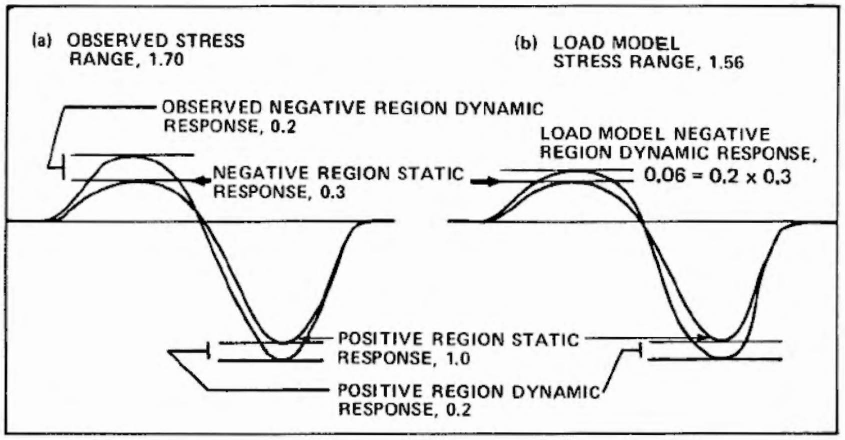

FIG. 11. Comparison of observed and load model stress ranges for a two-span continuous bridge.

mean dynamic amplification observed from all trucks whose average weight would be, perhaps, 300–400 kN. Figure 9 suggests that using this specified dynamic load alowance for a 700 kN OHBD truck will result in an overestimate of dynamic load because the dynamic response to suit a heavy truck will be much less than the mean. Similarly, using the specified dynamic load allowance for a light truck used for evaluation of a posted bridge might underestimate dynamic load. It might be more representative to specify a dynamic load allowance based on data only for the heaviest trucks, and use that with the OHBD truck to compute a dynamic load. This load would then be used for all trucks, irrespective of the weight of the truck actually used for design or evaluation.

For the shorter bridges tested, there was greater uniformity in mean dynamic amplification between the four test vehicles. This is expected, as force effêcts do not increase linearly with truck gross weight until the bridge becomes rather longer than the heaviest truck.

5.4 Negative and residual region effects

The data in Table 3 shows, in general terms, that mean dynamic amplifications of continuous bridges are approximately equal for both positive and negative regions, and the coefficients of variation are also approximately equal. The residual region dynamic response is also similar. This is evident simply by considering a truck modelled as a pulsating force on one span of an example two-span continuous bridge. The envelope of force effects due to bridge response in its modes of vibration will give, on average, equal effects for each span. However, the envelope of static effects is very different, as is well known. Dynamic load is presently modelled in OHBDC as an increase in live load. As Fig. 11 shows for the example two-span bridge, this load model underestimates the dynamic response expected in the negative region compared with the test result. The practical stress range, therefore, would be (1.7/1.56) times higher than by current design practice, which when cubed would give a 29% increase in fatigue damage. Uncounted strength, such as from curbs, barrier walls, and bracing, which acts under service loads, and other conservative features in design, ensures this is not a problem at present, though it may become a concern as fatigue increasingly controls design.

The load model suggested by Csagoly and Dorton (1973), a uniform positive and negative moment distribution added across the entire bridge, would address this observation for both simple and continuous spans.

5.5 Other observations

The vibration modes excited by a truck depended very much on the truck and its lane of travel. For simple spans, the first flexural and torsion modes were usually evident in the oscillograph records and in some multi-girder bridges transverse flexural modes could also be seen. For continuous bridges, especially the larger ones, various combinations of the first several flexural and torsional modes were seen. The first flexural mode was not always evident. The Fourier analysis technique previously described ensured that no modes were missed.

When test runs were made with two test vehicles side by side or in echelon on the bridge, the dynamic amplifications were generally reduced per vehicle, though there was insufficient data for reliable statistics. Unusual situations, such as trucks following very closely or passing in opposite directions, resulted in responses that were difficult to interpret. They could include significantly greater dynamic response than seen in the single vehicle test runs.

Runs by test vehicles following each other separated by one or two span lengths of a simple or continuous span generally resulted in dynamic amplifications

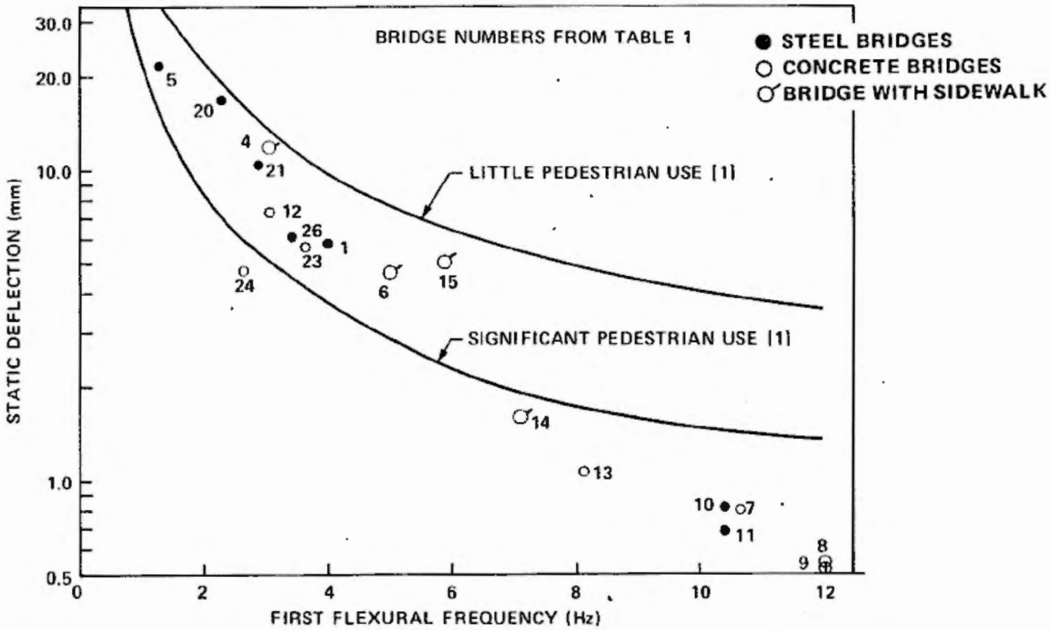

FIG. 12. Comparison of measured deflections at edge of bridge, adjusted to design load, with deflection criteria.

10–30% higher than those from an initially quiescent bridge. Such runs were included in the overall statistical data presented above.

The frequency match of vehicle and bridge was noticeable on the South Nation River bridge, whose 1.31 Hz first flexural mode was perceptibly excited by body heave of passing intercity busses. This was the only bridge that responded perceptibly to a bus.

6. Human response

During a test, technicians and others were asked to stand on the bridge and provide a subjective rating of bridge vibration due to passing trucks. The Reiher–Meister descriptors were used: not perceptible; slightly perceptible; distinctly perceptible; strongly perceptible; disturbing; and very disturbing. No training or calibration was given to the subjects. The only instruction was that they should use their own interpretation of the descriptors. All subjects were accustomed to vibration of bridges.

Subjects proved well able to distinguish significantly different vibration levels of consecutive trucks, but were not consistent in assigning an absolute rating to a particular acceleration level. A high acceleration followed by a lower one usually resulted in a higher rating for the second than if it had occurred after a lesser acceleration, and two consecutive accelerations of approximately equal level often did not get the same rating. There was also some evidence of learning during the test, particularly as far as expectation of a rating for a test vehicle.

The threshold of perception was in the range of $0.015g$–$0.025g$. The slightly, distinctly, and strongly perceptible ratings had mean acceleration of $0.039g$, $0.052g$, and $0.076g$, respectively. Occasional disturbing ratings were recorded, generally from lightly loaded trucks, but surprisingly none was attributed to Cavanville Creek bridge, which consistently gave accelerations in the $0.15g$–$0.25g$ range. This bridge was excited particularly in the 9–12 Hz range, which apparently coincided with a frequency of the author's jaw and was found somewhat annoying!

These acceleration ratings could not be compared directly with the criteria for pedestrian serviceability. The deflection criteria for pedestrian serviceability are compared in Fig. 12 with results from the tests, factored from deflection at the edge of the bridge produced by TV3 in a traffic lane by the specified $0.8 \times LL \times (1 + I)$. None of the bridges tested had significant pedestrian use, and while some scatter is evident, the criteria do not appear to warrant other than minor adjustment. This comparison includes the often significant stiffening effects of curbs and barrier walls. It is possible that neglect of these contributions to stiffness in design may lead to unnecessary increase in main longitudinal member sizes.

Heaving of the deck while the Gull Lake Bridges were vibrating was visually perceptible. No other bridge exhibited visible vibration.

7. Conclusions

During 1980 MTC conducted dynamic tests of 27 bridges to gather additional data in support of the dynamic load and vibration provisions of the Ontario Highway Bridge Design Code. All tests were conducted in a standard manner. Data were obtained from 100 or more truck passes for each bridge so that statistics of dynamic response could be computed to arrive at the load factors of a limit states code.

The vibration characteristics of bridges can be reliably determined by the Fourier analysis methods used in these tests. It is more difficult for shorter spans, or if the bridge consists of several simple spans. Bridges of conventional timber construction apparently did not vibrate.

Bridges typically are considerably stiffer under the service loading of a single truck than is assumed for design. Barrier walls, curbs, the deck, and secondary members all may contribute stiffness not considered in the design process, even if they are not continuous along the structure. A fully effective section responds with different deflection and stress distributions under a single truck loading than the ultimate load structural model. It appears that economies might be obtained in bridges whose design is governed by fatigue or deflection, serviceability limit states with single vehicle loading, if a structural model representing bridge response at service loadings would be used.

The data generally confirmed that the form and values selected for dynamic load and serviceability provisions in the first edition of OHBDC were adequate, and only required minor revision for the second edition. Other observations made from the data suggest a more representative form for dynamic load allowance might be possible. This will require further careful examination of the data, some analysis, possibly some additional testing, and an understanding of the effects on other areas of the Code.

Acknowledgements

This work was initiated by P. F. Csagoly, former Head of Structural Research at MTC. Testing was conducted by Z. Knobel of Structural Research and staff of the MTC Research Laboratory. It has benefited from discussions with other member of the OHBDC Sub-Committee on Dynamics and Vibration: R. Green (Chairman), T. I. Campbell, and M. S. Cheung. The author gratefully acknowledges the efforts of these and all others who contributed to this large project of testing and data analysis.

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RECEIVED FEB 25 1986 MATERIALS & MECHANICS DIVISION

NATIONAL SCIENCE FOUNDATION

PROJECT SUMMARY

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DIRECTORATE/DIVISION ECES	PROGRAM OR SECTION Systems Engr. for Large Structures	PROPOSAL NO.	F.Y.

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TITLE OF PROJECT

Bridge System Identification from Modal Testing

TECHNICAL ABSTRACT (LIMIT TO 22 PICA OR 18 ELITE TYPEWRITTEN LINES)

The use of impact testing with large instrumented hammers and accelerometers has not been used extensively on large civil engineering structures. Recent developments in modal testing techniques and in partial eigenvalue solution and sensitivity relationships are such that it is now feasible to perform system identification of large civil engineering structures to obtain estimates to physical parameters of finite element models from modal information obtained from impact tests.

Three applications of this methodology will be pursued including tests on existing bridges.

(1) Evaluation of structural integrity through comparison of measured dynamic properties with other bridges and finite element models.

(2) Estimation of parameters associated with energy dissipation, foundations effects, and the interaction of the superstructure with the abutments, from measured dynamic properties of the bridge and a finite element model.

(3) Prediction of the effects of proposed structural modification on the dynamic characteristics of a rehabilitated bridge by observation of modal information before and after structural modification.

Should this approach give good results for these three applications on the bridges tested, this study could have important economic consequences, particularly for application to the repair and rehabilitation of the deteriorated infrastructure in the country, bridge structures being the most notorious. The procedure requires easily transportable equipment, would necessitate minimal disruption in traffic flow, and could easily be incorporated in existing inspection procedures.

The data reduction to modal format requires fairly sophisticated modal testing equipment which is common in the aerospace and mechanical expanding field, but which has not yet been used extensively in civil engineering. The structural system identification algorithm used to determine physical parameters from this modal information uses efficient sensitivity relations based on partial knowledge of the calculated modal properties.

NSF Form 4 (7-83)

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PROFESSIONAL ASSOCIATIONS

- Ordre des ingenieurs du Quebec, 1974 - 1986
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UNIVERSITY DEGREES

B.S., Civil Engineering, University of Vermont, 1968.

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EXPERIENCE

- Vermont Highway Department and various engineering consultants, summers from September 1964 to May 1968

- General Electric Armament Department, Burlington, Vermont, Engineer May 1970 - September 1970 and May 1968 - September 1969

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- Evans Scholarship, (University of Vermont), Sept. '67-May '68
- Tau Beta Pi and Chi Epsilon (University of Vermont)
- Gold Medal in Engineering from American Military Engineers (University of Vermont)
- Edward Haight Phelps Memorial Prize (University of Vermont)
- Graduated Cum Laude (University of Vermont)
- Society of the Sigma Xi (Princeton University)
- Postdoctoral Research Assistantship, (Columbia University) Sept. '73-June '74
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COMMITTEES:

- Canadian National Committee on Earthquake Engineering, (1976-1982)
- Third Canadian Conference on Earthquake Engineering, Montreal, Organizing Committee, (1979)
- 22nd International Conference on Integrated Civil Engineering Systems ICES, Montreal, Organizing Committee, (1979)
- Faculty Affairs Committee, University of Vermont 1985 - 1986

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Beliveau, J-G, "Current Modal Testing Proceedures for Base Excitation of Flexible Structures", invited presentation for AIAA mini symposium on Large Aerospace Structures; Dynamics and Control, Cambridge Mass. April 23, 1986.

MASTERS THESIS SUPERVISION

Favillier, Michel, "Identification des parametres dynamiques d'un batiment soumis a des charges cycliques", Master's Thesis, November 1979.

Chater, Samir M., "Sensibilite et estimation des parametres dynamiques d'une batisse a partir de mesures modales", Master's Thesis, November 1981.

Simard, Bruno, "Edifices soumis a des charges sismiques; analyses, experimentation et design", Master's Thesis, March 1982.

Soucy, Yvan, "Analyse dynamique de grandes structures par optimisation et synthese modale", Master's Thesis, July 1982.

Nguyen, Thanh Son, (Co-director, Lefebvre, G.), "Simulation numerique du comportement dynamique de depots de sol", Master's Thesis, August 1982.

Noiseux, R. (Co-director, Lefebvre, G.) "Comportment d'un till sous chargement cyclique", 1982.

Ameziane - Hassani, H. "Analyze d'instabilitie en bande de cisaillement de materiaux non-standard par ondes stationaires" 1986.

Lefebvre, D. "Effect de l'energie potentielle dans le comportment statique et dynamique des structures" 1986.

Rouillard, J. "Instabilite et comportement dynamique des echafaudages", ongoing

Lintermann, C. "Parameter Estimation of Cable-Stayed Pedestrian Bridge from Modal Information through Impact Testing" ongoing

Rong, H., "Damping Synthesis Using Complex Substructure Modes" ongoing

Dolhon, A. "Efficient Sensitivity Relations in Structural System Identification" ongoing

DOCTOR'S THESIS SUPERVISED

Thomas, M. (Co-director, Massoud, M.) "Identification des parametres physiques d'une structure amortie par excitation appropriee dephasee", 1986.

UNDERGRADUATE COURSES

Title of Course		Textbook	Author
- Strength of Materials	(1)	Introduction to the Mechanics of Solids	Popov
	(2)	Mechanics of Materials	Beer & Johnston
- Advanced Strength of Materials		Introduction to the Mechanics of Solids	Popov
- Determinate Structures		Structural Analysis	Laursen
- Indeterminate Structures		Calcul des structures hyperstatiques	Ellyin
- Wood Design		Notes	Canadian Wood Council
- Reinforced Concrete Design	(1)	Reinforced Concrete Design	Wang & Salmon
	(2)	Design of Concrete Structures	Winter & Nelson
- Advanced Structural Design		Structural Steel Design	McCormack
- Computer aided Design of Structures		Structural Design by Computers	Wright

GRADUATE COURSES

- Probabilistic Methods	Probabilistic Methods in Civil Engineering	Benjamin & Cornell
- Energy Methods	Solid Mechanics Variational Approach	Dym & Shames
- Advanced Structural Theory	Computer Methods in Advanced Structural Analysis	Wang
- System Identification Concepts in Structural Dynamics (UCLA, 1976)	Solving Least Squares Problems	Lawson & Hanson
- Structural Dynamics	Structural Dynamics	Paz

RESEARCH GRANTS: Sherbrooke/Vermont

Title	Principal Investigator	Funding Agency	Amount	Span
Structural System Identification	J-G Beliveau	National Science and Engineering Research Council of Canada	19,000/yr	1975-1984
Stability of Metal Scaffolds	J-G Beliveau & K.W. Neale	Institute de Researche en Sante et Securite du Travail du Quebec	250,000 3 yrs.	1984-1987
Critical Structural Behavior	K.W. Neale F. Ellyin	Formation de chercheurs et l'aide a la recherche	34,000/yr	1975-1985
Behavior of Soft Sensitive Clays	G. Lefebvre	Formation de chercheurs et l'aide a la recherche	35,000/yr	1979-1986
Satellite Dynamics	M. Massoud	Minister of Communications of Canada	30,000/yr	1980-1986
Dynamic Testing of Civil Engineering Structures	J-G Beliveau	University of Vermont Committee on Research Scholarship	9000	1985-1986
Modal Testing Equipment	J-G Beliveau	Department of Defense	98,167	pending
Modal Testing Equipment	J-G Beliveau	National Science Foundations	77,385	pending

REFERENCES

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Prof. R. H. Scanlan, Department of Civil Engineering, The John Hopkins University, Baltimore, Maryland 21218-2699.

Professor Guy Lefebvre, Department of Civil Engineering, Universite de Sherbrooke, Sherbrooke, Quebec, J1K 2R1

Professor Mounir Massoud, Chairman Department of Mechanical Engineering, Universite de Sherbrooke, Sherbrooke, Quebec, J1K 2R1.

BRIDGE SYSTEM IDENTIFICATION
FROM
MODAL INFORMATION

J-C. Beliveau
University of Vermont

Table of Contents

I. Current Barriers

The barriers to research competitiveness which I feel have limited me during my first year at the University of Vermont are the following:

1) Equipment
2) Graduate student funding
3) Technical support
4) Isolation

1) Equipment

For the last eleven years while in Canada, my research in structural system identification has largely been of a numerical nature. I did perform tests on a steel building while at The University of California at Los Angeles for three months in 1976, using equipment borrowed from the California Institute of Technology. I also performed wind tunnerl tests on a suspension bridge model at the Federal Highway Administration while at Princeton University. Otherwise, I have relied on data from the National Research Council, in Ottawa for a twelve story concrete building, and the Department of Communications, also in Ottawa, for a component of a communications satelite. I also did shaker table tests and modal testing while at Sherbrooke, using modal testing software developed by Structural Measurement Systems (SMS) and a Bruel and Kjaer two channel spectrum analyser.

I now wish to integrate structural testing to my research, concentrating on testing of full-scale structures. Recently, the University of Vermont granted me a small grant with which I was able to acquire the necessary accelerometers, impact hammer and tape recorder. I have made temporary arrangements with the General Electric Company, here in Burlington, to borrow their GENRAD #2515 spectrum analyzer and modal testing software, which I have used in the past. This equipment is essential for the work proposed here and has been included in the budget. I also have applied to DoD and NSF for this equipment and their decision is pending.

2) Graduate Student Funding

I currently am supervising three masters degree students in civil engineering at the University of Vermont:

C. Linterman - Modal Testing of Cable-Stayed Pedestrian Bridge
A. Dolhon - Modal Sensitivity and Parameter Estimation
H. Rong - Damping Synthesis using Complex Modes

All three are supported in this interim period by University of Vermont internal funds during a nine month period. C. Linterman was also supported over the course of the summer by funds granted me by the University of Vermont Committee on Research and Scholarship. These graduate students working with me interact with mathematicians, statisticians, mechanical, electrical and civil engineers, in the areas of linear algebra, statistical inference, vibrations, signal analysis, and structural analysis respectively.

My previous students at Sherbrooke have found excellent job opportunities in industry and government positions in Canada and in Europe. Given appropriate funding, I feel I could attract qualified students from both inside and outside the University of Vermont to work with me. Although the University is supporting these three students in this transition period, it is expected by my research grants and/or student fellowships will eventually support these students, two of which would work on this project.

3) Technical Support

There are seven civil engineering and eight mechanical engineering faculty in the Civil Engineering and Mechanical Engineering Department at the University. There are only two technicians assigned to these fifteen full time faculty, and two secretaries. In the civil engineering department at the University of Sherbrooke, where I was before coming to Vermont, there were two secretaries and six technicians, all university positions, for the fifteen faculty. In addition, two secretaries and three to four additional technicians were supported by research grants.

The modal testing hardware and software I will need for this work requires a fairly sophisticated knowledge in my opinion. Though a technician may be able to grasp the operational aspects of the apparatus, an engineer, with appropriate training in this area would be much more qualified to participate in the testing, data acquisition, data reduction, and finite element aspects of the project, as well as in the general maintenance of the hardware and updating of the software. It is expected that twenty five percent of his annual time will be required, primarily during the summer months.

4) Isolation

Burlington, Vermont is a relatively isolated city and is not a major industrial center. In addition, though there are quite a large number of researchers in the United State, working on infrastructure considerations, very few are involved on system identification aspects and full scale testing of structures. I have collaborated with a number of individuals, both in Canada and in the United States, and would like to have these and others give seminars at the University of Vermont. At the University both Profs. J. P.Laible and M. S. Hundal have interests in finite element modeling and modal testing.

Possible speakers with whom I have collaborated previously and tentative topics are enumerated below:

Subject	Tentative Speakers
Modal Testing	F. Vigneron, M. Novak, R. Vaicaitis
System Identification	G.C. Hart & J.T.P. Yao, M. Massoud
Safety Evaluation	J.T.P. Yao & M. Shinozuka
Full Scale Testing	R.H. Scanlan & A.M. Abdul-Ghaffar
Material/Connections	R. Redwood, D. Mitchell, K.C. Johns

Of course, the Vermont Agency of Transportation will be consulted during this work. In addition a New England Surface Transportation Infrastructure Consortium which has recently been formed has an interest in bridge evaluation. Dr. R. Dorton, currently editor of the Canadian Journal of Civil Engineering, for which I am one of the associate editors, has long been active in the Ontario, Canada bridge testing program, and I will consult him and members of his ministry on their ongoing work in this area. Profs. M. Novak of the University of Western Ontario and R. Vaicaitis of Columbia University, have both recently acquired modal testing capability, with a view to applying it to civil engineering structures. I collaborated with M. Massoud while at Sherbrooke and would like to pursue certain aspects with him which are related to the proposed work.

Having been educated mainly in the United States, I returned to my native Canada for eleven years, at the Universite de Sherbrooke. This was an extremely rewarding professional experience, in which I worked in the areas of structural and soil dynamics, including soil-structure interation of nuclear power plants and dynamic testing of satellite structures. Though I published in well respected journals, and gained national prominence in Canada, I feel that my competitiveness in this country might have been better served had I remained in the United States. These seminars would permit me to maintain the impetus of my previous work. In addition, they would give both my colleagues and students varying points of view on bridge testing, modelling, system identification, and safety considerations.

Depending on the outcome of the three year study, I anticiapte the additon of a younger faculty member with whom I would pursue related aspects of structural system identification. The structural system identification research area in which I have evolved over the last twelve years is interdisciplinary in that both numerical analysts, and experimental analysts are involved and both civil and mechanical structures are considered with specific assumptions, frequency range, etc. It is a tremendous area for research, with important implications in evaluation of structural integrity, stability and strength.

II. Research Objectives

Bridge structures are typical of general infrastructures in this country. Due to their age and loading they have deteriorated, over time. But, is one particular bridge capable of supporting current or projected loads from a structural viewpoint? If not, can it be repaired or must it be replaced? What will be its behavior should it be rehabilitated?

The objective of the proposed research is to develop and implement a testing procedure to assist the structural engineer in making such important economic decisions. Based on relatively simple tests combined with appropriate numerical models of the bridge, it is hoped that a quantitative assessment of the structural integrity of existing bridges will be possible.

To do this it is suggested that dynamic rather than static measurements be performed. This involves little equipment which is completely transportable. This data is then analyzed by a modal testing system. Finally physical parameters are determined to have the calculated modal parameters agree more closely with hose obtained from the measurements.

From this information and the parameters obtained, the effect of proposed structural changes on the bridge behavior may be deduced once incorporated into a finite element program. Ultimately, this would yield realistic estimates of current capacity and the capacity for the structure for given modifications.

III. Plan of Work

III.A Introduction

The National Science Foundation has recognized the problems of the bridge infrastructure in this country in a recent workshop on the importance of evaluating the performance of bridge structures and in their strengthening and in research needs(1). Bridges are massive and large, and are composed of many members. Static testing of their behavior requires large loads and substantial equipment to monitor the response. Material properties change over the life of the structure as does the loads imposed on it and the way it is used. Dynamic testing apparatus is less cumbersome than that used in static tests, requires less time, and yields reliable information. For these reasons it is also applicable to inspection procedures.

Modern numerical methods of structural analysis have evolved to a very sophisticated level for which fairly complex physical behavior, both static and dynamic, may be adequately modelled. Parallel to this dynamic testing procedures have been improving as well. System identification is concerned with comparison of these numerical results and measured observations and the modification of parameters of the numerical model to more closely match the measurements in a systematic manner. Depending on the correlation of the measurements and the model, a level of confidence in the model and its parameters is obtained. Results may also be used in obtaining a quantitative evaluation of the structural performance in stiffness, strength, or stability, based on the parameters estimated, and on the prediction of anticipated behavior for given design changes, based on the sensitivity calculations utilized in the identification process. Results are also useful in determining whether current design codes are met, and in determination of load capacity.

Normally, complicated relationships exist between the observed dyanmic quantities and the physical parameters. Generally limited information is available either in quantity, be it in time history measurements, or in frequency range. The number of mode shapes measured and/or resonant frequencies may be a small percentage of those obtained from the corresponding finite element model. Since different measurements have various precisions, data must be weighted accordingly. Sensitivity calculations must be based on analytical formulae rather than on numerical discrete parameter changes. They should also be based on limited information. Initial estimates of the parameters should also be weighted relative to each other as well as to the measured data within a Bayesian framework. In order to correlate measurements with bridge strength, the parameters determined must have physical sense such as mass and stiffness rather than mathematical, such as elements of matrices, or modal, such as resonant frequencies. Procedures for parameter estimation should be able to incorporated these considerations.

The usual assumption of small vibratory motion about an equilibrium position is made. Lagrange's equations are then expressed as a linear second order system of ordinary differential equations with constant mass, stiffness and damping characteristics. For arbitrary viscous damping, numerical procedures are used to integrate these or to solve the corresponding eigenvalue problem. The mass characteristics are explicitely physical, (density and volume) whereas the stiffness are normally obtained numerically from the physically representative geometry and material characteristics. Damping, assumed to be viscous, has a physical interpretation in the form of experimentally obtained modal damping ratios. The response, whether in time or frequency, may be represented by the mode shapes of vibration, whether they be real normal modes or complex damped modes.

Time histories, frequency based response, wave propagation velocities and modal characteristics represent four forms of data normally available from dynamic testing procedures. Time histories result from free vibration, typical of impact or step relaxation tests for initial velocity or displacement respectively. They are also obtained as correlation functions or as complex exponential time series through Fourier inverse relationship of the cross spectral densities and frequency response functions respectively.

Forced response also generates time history data, but both the forces and the response must be measured, thus making them impractical when a large number of degrees of freedom are involved such as in structural dynamics. Sinusoidal loading or random excitation yields data in a given frequency range. Single or multiple shakers may be used in either a sine sweep mode or in a random mode. Measurements under ambient vibrations are easier to obtain. The second type of frequency based data are spectral densities which is limited to random excitation. Both require long time records because of averaging and the low frequency content of the signals.

For continuum modeling, rather than for discrete structural systems, measurements are of the wave propagation velocities. The damped natural frequencies, modal damping ratios and the mode shapes, all limited in number, form the modal type of information from dynamic testing.

For purposes of this proposal only nondestructive dynamic testing techniques will be investigated. Data, whether in the form of acceleration in time or response in frequency will be converted to modal information which will then be treated as the basic data in the parameter estimation algorithm. Wave propagation techniques and other similar methods useful in determining local properites will not be pursued since they have little to do with the overall bridge capacity, although, they are necessary for local material evaluation. Also, their use is fairly common.

From the modal information in the form of damped natural frequencies, corresponding modal damping ratios, and the associated mode shapes, physical parameters related to mass or member stiffness will be determined based on a nonlinear least squares technique incorporating both the modal information and a priori information of the parameters.

This investigation will limit attention to the following:

1) dynamic, rather than static testing procedures,
2) global modal characteristics associated with low frequency modes of vibration rather than local material properties obtained from wave propagation methods,
3) physical rather than numerical parameters,
4) efficient analytical representation of eigensensitivity relations rather than numerical differentiation,
5) discrete finite element models as opposed to continuous representation, and
6) the nonlinear least squares technique of system identification based on modal information.

III.B Structural System Identification

Though system identification has long been used in electrical engineering it is relative recent in mechanical and civil engineering applications(2,3,4,5). Modal information is often treated as data in many applications(6). In this situation efficient sensitivity relationships have been developed which depend only in the particular eigencharacteristics for which sensitivity is being sought(7). This is particularly useful in structural systems where interest is limited to the low frequency range, a few physical parameters are to be estimated, and the matrices are of large dimension. These three conditions apply to the study of bridges.

The relationship of structural system identification to the reliability and safety assessment of existing structures have been documented(8). Many full scale tests are based on static loading and required extensive experimental programs(9).

Dynamic testing of full scale structures has developed tremendously(10) and numerous government agencies have ongoing testing programs or have performed such tests on bridges(11,12,13). The equipment used to monitor dynamic motion is much less cumbersome than that used for static testing. In addition, only relative motion is needed for mode shapes, rather than absolute displacements required for static load testing. Partial data may be used including just frequencies and not mode shapes, and only some of the degrees of freedome(4) need be measured. Finally, modal damping ratios are obtained though they are for small levels of motion. It is expected that the damping ratios will be larger for increasing levels of motion amplitude.

The raw data is not, in general, in modal form, but will normally be either time histories of structural response to known or unknown dynanmic loads, or, secondly, may appear as frequency-based data such as frequency response functions or spectral densities. The three types of frequency data normally investigated are:

1) Spectral density of random vibration response to transient wind
2) Frequency response for a single exciter operating in a sine sweep mode
3 Frequency response function obtained from impulsive loading.

Under certain assumptions these may then be reduced to modal format including damped (or undamped) natural frequencies, modal damping ratios and real normal (or damped complex) mode shapes. Because of the relatively low frequencies and Fourier transform relationships, an average of a number of sample records is required for random response. Also, long time records are required for the first method, though it is less demanding in time and equipment than the other two techniques. The second method is the most demanding from an experimental point of view since a shaker (or multiple shakes) is required. It is not very practical because of the frequency range of interest and the low force level typical of mechanical exciters.

The third technique involving an impact load is much simpler than the step relaxation methods, but has not been used much on civil engineering structures. The impact hammer yielding a frequency response function over a relatively large frequency range has been successfully used on mechanical structures. We propose to investigate the applicability of this last technique to bridges.

Most of the laboratory controlled dynamic testing procedures yield data which is frequency based. This is true for sinusoidal or random excitation either with shakers or under ambient environmental loads such as wind or normal traffic. Both the free vibration response and the frequency response functions may be expressed in terms of the modes of the structure which may be real or complex, depending on the type of damping assumed(14,15,16). The inverse Fourier transform of the frequency function yields a sum of complex exponentials in time for the motion of each of the degrees of freedom. Furthermore, for multiple frequency response functions, complex mode shapes may be extracted either for excitation with single exciter or for base excitation(17). This approach is based on Prony's early work(18). It has recently been extended for sequential estimation(19). The procedure determines both the exponents and coefficients of a time seres which is a sum of complex exponentials in time.

Once the modal parameters are obtained, physical parameters may be estimated using this modal information as data in a second stage, with physical parameters now being the unknowns. This two stage approach to system identification has tremendous potential due to the relative simplicity of the testing and data acquisition at the first stage and the efficient sensitivity relations which would result in most situations.

III.B.1 Complex Exponential

The free vibration is a sum of complex exponentials in time. The inverse Fourier Transform of the frequency response function is also of this form. Though this approach is sensitive to noise in the time signal, relatively good modal data is obtained for the lowest modal characteristics, those associated with the lowest frequencies(20,21). In addition, for noisy data a relatively small portion of the high emplitude portion of the signal, due to an impulse, for instance, may be used either in batch or sequential form. current software yields complex modes, and approximate real normal modes from phase relations.

III.B.2 Nonlinear Least Squares

The method of nonlinear least squares has been used to determine parameters for time-based data of the free vibration motion of a scale model of a suspension bridge(22), the frequency based forced response of a four story steel office building(23), modal data of a twelve story reinforced concrete apartment building(24), and to satellite structures(25). It is applicable to modal data which has a complex relationship with physical parameters. Constraints on the parameters may readily be incorporated in an optimization algorithm, as can initial estimates to the parameters.

III.B.3 Partial Eigenvalue Solution

The sensitivity of the modal characteristics including the mode shapes, may be calculated from partial knowledge of the eigenvalues and eigenvectors rather than full knowledge as had been done earlier(7). Thus partial eigenvalue routines such as the subspace iteration modification of the classical power method(26) may be used as well as those based on a modified Rayleigh quotient(27).

III.C Methodology

The methodology to be investigated involves five basic steps.

1) Impact testing and ambient vibration monitoring.
2) The determination of frequency response functions of the measured response.
3) The reduction of the frequency based information to modal information.
4) The efficient numerical calcualtion of the sensitivity of the modal information to changes in the physical parameters.
5) The implementation of a general structural system identification technique for physical parameters from modal information.

The approach is planned to supplement current local inspection techniques and not to replace them as these will still be needed to evaluate local material parameters. A global quantitative structural evaluation is obtained by the procedure outlined here.

III.C.1 Impact Testing

The use of impact hammers is quite common for mechanical components in the laboratory. We wish to adapt this technique to large civil engineering structures tested in situ. Accelerometers will be used to monitor the response, which will be recorded on a tape recorder. Of course, the hammer will be such that large forces are generated and the equipment must operate in the low frequency range. The response of the bridge to ambient vibration, due to either wind or traffic, will also be of value since resonant frequencies may be obtained from this relatively simple instrumentation. The impact test will be used to confirm these frequencies, determine damping estimates and mode shapes as well, once it is processed. In both the transient and impact testing, the equipment used needs no batteries, is lightweight, and reliable. This makes is applicable to in situ testing, even on a regular inspection basis, to monitor the change of natural frequencies over the life time of the structure, for instance. Different frequency content may be obtained by using different heads, and members suspected of having deteriorated may easily be instrumented individually to determine local dynamic characteristics.

III.C.2 Frequency Response Functions

Dual-channel spectrum analysers have long been used in mechanical engineering applications as well as in structures, primarily for aerospace applications. They may be used to establish natural frequencies from ambient vibration, and more importantly frequency response functions from impact testing. Many analysers have been upgraded to multi-channel operation, rendering the data reduction step much less time consuming. Also specific windows are already incorporated, and they operate in the low frequency range. The use of impact testing requires shorter time records than the corresponding ambient vibration tests because of the relatively small number of averages needed.

III.C.3 Modal Information

The recent development of complex exponential algorithms has revolutinized modal testing. It is relatively routine in hi-technology areas to determine modal parameters, i.e. natural frequencies, modal damping ratios, and associated mode shapes, be they complex or normal, from frequency response functions. Depending on the number of tests performed, a large amount of information is reduced to a countable number of modal quantities. The basic steps used have been outlined in a previous paper(17).

III.C.4 Sensitivity Calculations

Because of the sensitivity relations, which are based on the sensitivity of the mass, damping, and stiffness matrices, numerical procedures for elements of local member stiffness parameters are relatively easy to implement since the global stiffness matrix is the sum of the local member stiffnesses. Effects such as shear energy, braces and secondary elements, additional mass, all affect the modal characteristics, as does changes in boundary conditions or fixity at the connections, supports, and in the soil foundation.

Though finite element models of large structures normally involve a large number of degrees of freedom, relatively few physical parameters, such as density, geometry, material properties are involved. Modern numerical algorithms used to calculate dynamic properties are based on the partial solution to the eigenvalue problems, which would otherwise require a large computer capacity both in time and memory. They determine information in the low frequency range. Secondly, efficient numerical calculation exist to calculate sensitivity of modal parameters with respect to physical parameters. It is believed that the modal information, natural frequencies, mode shapes, and damping are sensitive to deterioration and to parameters sought in the identification step. Many previous studies have considered only frequencies, or have looked at transient response. The use of mode shapes and damping should be useful in locating damaged areas as well as physical parameters.

III.C.5 Parameters Estimation

Energy dissipation is a complex phenomena, which depends on many parameters, in particular the levels of motion. In the context of this proposal, the physical damping parameters will be modal damping ratios, rather than based on first principles of physics. Thus, it will not be treated as an unknown physical parameter at the structural system identification step, but rather the values obtained from the modal information step, based on levels of motion of the experiment, will be used. For higher levels of motion, the energy dissipation will normally be increased.

Mass, and stiffness parameters of a finite element model are considered to be physical parameters. Many experimentalists are content with the modal information and most structural tests stop at this level. For purposes of structural evaluation or for the determination of the structural behavior for a given structural modification to the existing structure, be it a small repair or a major rehabilitation, modal information is not always sufficient. A model based on physical considerations is needed in many instances, such as when a particular structure has been targeted for further investigation. Modal information may, however, be quite useful in the initial stages of structural evaluation, for instance in comparing natural frequencies to those for similar bridges. Based on these initial findings, it may be decided not to pursue the sensitivity or system identification steps for a given bridge, which has been judged adequate based on these preliminary tests.

Though a large amount of data was initially available either in the time domain, or in frequency, the previous steps have reduced greatly the number of data to modal information. There may be more unknown parameters than data. Thus, the structural system identification procedures must be able to integrate initial estimates to the parameters along with the modal information. Secondly, the finite element model may have a large number of degrees of freedom, but observation at only a few of these is available. This fact should also be incorporated in the mehtodology. Thirdly, efficient sensitivity calculations must be implemented as well as efficient partial eigenvalue routines. Lastly, modal information is often limited in frequency due to limits on the equipment used. Only a few of the mode shapes and resonant frequencies may be observed. The procedure used to estimate parameters will allow for these facts.

III.D. Bridges to be tested

During the three year study we expect to concentrate on the methodology presented herein, rather than the testing of large numbers of bridges. With this in mind then we propose to investigate three sub-projects

1) Connection and foundation effects of a cable-stayed bridge.
2) Effect of bridge rehabilitiation on dynamic characteristics.
3) Highway bridge inspection from modal data.

III.D.1 Cable-Stayed Bridge

A pedestrian cable-stayed steel bridge has recently been built in the Burlington area, within easy access to the University of Vermont. One graduate student has monitored the motion of this bridge under ambient wind and performed impact tests on the deck. He is in the process of obtaining the measured mode shapes and resonant frequencies using the GENRAD modal testing system on loan from the local General Electric plant.

Because of structural considerations, and mass distribution, this bridge has nearly identical resonant frequencies in vertical and torsional motion. The measured natural frequencies compare very well with those of a finite element analysis for both types of motion. Damping is small as would be expected at these levels of motion.

In this proposal we intend to perform additional tests to determine the effect of the interaction of the superstructure with the abutments, the effect of the soil on the dynamic characteristics of the bridge, and realistic levels of damping. Stiffness and mass parameters, and equivalent springs at the supports will then be estimated based on these measurements.

III.D.2 Bridge Rehabilitation

Many bridges are in various states of disrepair in Vermont as well as the rest of the nation. It is sometimes decided to repair existing bridges. Though a specific bridge has not yet been chosen, the purpose of this sub-project is to monitor the dynamic characteristics of a particular bridge both before and after a major structural modification has been made. These tests combined with proper system identification and finite elements will give us an indication of the applicability of these techniques in predicting the dynamic behavior of the bridge as modified.

III.D.3 Highway Bridge Inspection

The two previous projects each dealt with one bridge. Fairly extensive testing, modeling and parameter estimation techniques are to be used in those two cases. The purpose of this, the third sub-project, is to investigate the feasibility of using impact testing to arrive at a relatively simple quantitative evaluation of a particular bridge within an inspection routine, much as is done currently for local material properties, connections, etc. The transportability of the equipment to be used at the site makes this technique applicable in such instances. It would not necessarily be practical to always have to model each bridge tested by a finite element model, nor to determine its parameters. This test is meant to help target bridges which perhaps should be investigated further, much as current inspection techniques are being used.

The comprehensive study of twenty seven highway bridges in Ontario, Canada by Billings(11) revealed that the first frequency of vertical motion had a particular relation with the span length and was independent of whether the bridge was made of concrete or steel. The logarithm of the frequency had a linear relationship with the logarithm of the span length. Thus, a relatively simple criteron to evaluate structural integrity of a particular bridge, should this prove to be true in Vermont as well, is to determine whether or not the first frequency is within a tolerance level of the relationship. This could also be applied to higher frequencies,provided vertical frequencies could be distinguished from lateral and torsional frequencies, using mode shape information.

Of course an alternative criteron would be to monitor these measurements over the life of a particular structure. This, unfortunately cannot be done within a three year period. Billings(11) also mentions that resonant frequencies were not measured for timber bridges. This could be due to the high level of damping normally associated with timber structures. Perhaps, the frequencies were higher than the limits of the equipment used. Also the measurements were for test vehicles and not for impact loads. Nevertheless, it may be feasible to attempt the method proposed here to timber structures, such as covered bridges, typical of many secondary roads of this area of the country.

Attention will, however, initially be given to regular highway bridges, where the frequency will be measured from an impact test. Damping information of the various modes will also be taken. It may be necessary to monitor a number of points to distinguish the various types of mode shapes. This sub-project will only consider modal information and no attempt will be made to correlate the measurements with finite element models for the bridge tests. We plan to test a dozen highway bridges made of steel, concrete, or composite construciton within relatively access to Burlington.

III.E Conclusion

Modal testing procedures based on impact loads generated by a hammer are commonplace in the fields of mechanical engineering and aerospace structures. They have proved to be convenient in confirming numerical models and in assessing structural integrity of mechanical components.

The continued use of static testing methods for civil engineering structures has prevented a similar situation for these structures due to the cumbersome equipment required and the tedious testing program needed. The current state of our bridges obliges us to study alternative techniques to evaluate these structures, techniques which can be used in an inspection context, which are flexible and require equipment which is easily transportable, and which will interfere very little with traffic flow.

Through impact testing of full scale bridges, combined with modal testing procedures and system identification techniques for finite element models we hope to show that this approach

1) is useful in ascertaining which bridges should be investigated further by a simple modal test,

2) may be used in estimating parameters, such as damping, foundation flexibility and the effects of connections utilizing modal information and a finite element model in a system identification text,

3) may be used in predicting the structural behavior of a bridge given specific design modifications and a numerical model which has been adjusted to incorporate measurements on the existing bridge.

These are, of course, but a few of the topics which address the infrastructure problems in this country. Research needs encompass the above aspects and include others as well. Specific topics having much interest, but which are not covered here are

1) Damping estimation for large motion such as from an earthquake. It is expected, however, that damping at higher levels of motion will be larger than that measured.

2) Capacity evaluation at failure. The results obtained may be useful in a numerical analysis of the capacity, however.

IV. Significance of Work

The current state of deterioration and disrepair of the infrastructure of this country is alarming, bridge structures being the most reputed. Quantitative methods are needed to evaluate the structural integrity of these structures, which are often judged based on aesthetic or qualitative aspects. Since the overall strength of a structure is greatly influenced by the stiffness of its elements and connections, dynamic modal testing should offer a practical and efficient mechanicsm for determining physical parameters and/or appropriate modeling assumptions. Once these are obtained and validated as proposed here, a more realistic evaluation of the bridge capacity may then be determined from appropriate strength considerations. There is a need for damping measurements on bridges as NSF has pointed out in recent workshops, as well as on foundation interation(1).

The load capacity evaluation of a bridge structure, based on nondestuctive dynamic testing, rather than actual collapse loading of a prototype modal is, at best, approximate. Nevertheless, with the current modeling possibilities offered and current modal analysis dynamic testing procedures, an attempt should be made to determine physical parameters to more closely match observed behavior. The data must be easy to obtain, require little or no traffic interruption, and should be sensitive to the parameters being determined. The number of parameters to be estimated should be small in number and should have physical sense. Such a procedure will not eliminate the need for local testing procedures, useful for evaluation of material properties during normal inspections. It is, however, more representative of global bridge capcity than local methods.

V. Relation to Long Term Goals

I have been active in system identification techniques as applied to both mechanical and civil engineering structures since 1968. I first applied these techniques to estimating aerodynamic coefficients of missiles from flight data while at General Electric in 1969 and then for structural applications while a graduate student at Princeton(22) and a post doctoral research assistant at Columbia(6). The application to civil engineering structures in the last ten years while at the University of Sherbrooke has recently been summarized(4).

The basic premise of the parameter estimation algorithms used is that both the free vibration response and the frequency response functions are functions of the modal characteristics(14,15). Current modal analysis testing procedures utilizing the complex exponential approach to yield model information have been extended to base excitation used in both the nuclear and aerospace industries(17). Parameter estimation of physical structural parameters has been applied to a building with a limited number of mode shapes and resonant frequencies(24), a building from frequency response function obtained from mechanical exciters mounted on the roof(23), resonant frequencies of a satellite structure as obtained from step relaxation measurements(25). Damping parameters were determined from modal data in the case of real normal modes(6) and complex modes(21).

In addition to these applications of parameter estimation, I have worked on related problems of structural/soil dynamics.

1) frequency response formula for resonant testing(14)
2) eigenvector relations between first and second order formulations of equations of structural dynamics(15)
3) determination of partial eigencharacteristics of large finite element models by nonlinear optimization(27)
4) damping synthesis in substructure coupling(29)
5) aeroelastic behavior of suspension bridges(22,30,31)
6) structural stability evaluation by modal measurements(32,33)
7) soil-structure interaction of nuclear power plants(34)
8) non normal modes for site response studies(35,36)
9) surface wave propagation at the top of clay deposits(37)
10) material instability through wave propagation(38,39).

I feel that modern modal testing techniques combined with developments in partial eigensolution of the matrix equations of structural dynamics will revolutionize structural evaluation. The tests are simple and the numerical calculations to estimate physical parameters from modal information are no longer prohibitive from a computational viewpoint. I wish to apply these principles and techniques to full scale structures, bridges in particular.

VI Specific Outcomes

The structural evaluation of existing infrastructure facilities is of national concern, and has large economic implications in this country. Structural engineers are increasingly asked to perform such evaluations. Though many experimental techniques are available for local investigation, such as determination of specific material properties, joint behaviors, and effect of corrosion, there are no easily implemented experimental procedures to evaluate the global structural integrity of these existing structures. Often a qualitative assessment is all that is done.

The reticence to incorporate analytical tools, such as the finite element procedure, into the evaluations of these structures, based on observed behavior, stems from three basic facts: (1) the relative complexity of such models, (2) the large amounts of data that is obtained, and (3) the inappropriateness of many parameter estimation techniques to integrate the numerical calculations and easily obtained experimental observations. The intent of this proposal is to develop such a methodology, to implement it, and lastly to evaluate it.

Based on my past experience with parameter estimation from dynamic measurements, I believe this study will show the following:

1) Observation of motion under impact loading is a quick way of determining, based on comparisons with results of similar bridges, which bridges should be investigated further. It would be relatively simple and economical to integrate this to existing inspection procedures.

2) Large impact hammers used on existing structures will prove to be as practical in determining dynamic characteristics as small hammers are for mechanical components. The equipment used on site is easily transportable and will interfere very little with existing traffic. Further, it will prove useful in long term inspection of bridge structures.

3) The large amount of time or frequency data obtained may be converted to a small quantity of modal information by readily available spectrum analysers. This type of information obtained from routine inspection, will be useful in determining the general trend of the structure during its lifetime, and also in detecting degradation and deterioration of a structure, when compared to other structures.

4) For bridges tested with the hammer, partial data for both frequencies and elements of the mode shapes, may be used in determining existing physical parameters for a given mathematical model of the structure. From this information models of varying complexity may be evaluated, relative to the behavior observed experimentally, including the effects of connections and foundation interaction.

5) Based on the parameters of an appropriate model, the capacity of the bridge to withstand future loading may be determined in a quantitative manner. In situations where repair is deemed appropriate, different modifications may be assessed relative to each other by proper integration of the estimated parameters. This will be done for a particular bridge which must be repaired.

VII. Budget Justification

It is expected that his investigation will last three years. Two graduate students per year is sufficient to pursue this work, provided an engineer, responsible for the equipment and some of the programming aspects, is also involved.

I have worked in the area of structural system identification since 1970. Two colleagues at Vermont, J. P. Laible will be consulted in this area and M. S. Hundal in the area of modal testing. B. K. Stearns, at General Electric in Burlington, will be consulted regarding the GENRAD system.

Some of the numerical work will be done on personal computers utilizing software I have developed. I do expect to need larger computer requirements, however, for the system identification, and sensitivity calculations.

M. Drew is a secretary in the CEME Department. Two graduate students are currently supported by the faculty and will eventually be supported on this grant. D. Truesdell, a mechanical engineer, is interested in computer applications. While working at the University on another project he is enrolled in my graduate course in structural dynamics and a graduate course in finite elements given by J. P. Laible. He is very capable and is familiar with the computers on the campus. He mentioned to me an interest in graduate work in this area at a later time.

The equipment asked for includes both the GENRAD hardware, a #2515 four channel spectrum analyser, and the SDRC software on modal testing. I have worked with this system previously(17), and found this system superior for modal testing to the S. M. S. and B&K combination. Also, we have VAX computers available in the faculty, and General Electric is using their GENRAD system in combination with VAX computers. This will permit me to collaborate with them on certain aspects of this work. The work proposed cannot be done without this equipment.

Finally, a budget item for visits and visitors is included. This item will cover honoria and expenses of visitors to Vermont. In the last two years of the three year proposal, I expect to allocate some of this budgeted amount to distinguished visitors who will be working with me over extended periods, possibly for a portion of a sabbatical year.

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IX. Impact of Proposed Research

The sad state of disrepair and degradation of the civil engineering substructure in this country obliges us to reevaluate the procedures used to evaluate structural integrity of existing structures. Though many inspection techniques are available to arrive at local material properties, there are currently no easily implemented techniques to evaluate the structural integrity of bridges.

The use of large impact hammers combined with modal testing equipment and system identification techniques may prove to be an efficient methodology to do this. In particular this methodology has these characteristics:

1) Equipment is easily transported to the site, needs no power, and will minimize traffic interruption. It can, thus, be readily integrated into current inspection procedures.
2) Data on the overall structure as well as specific structural components can be readily obtained, and different frequency ranges may be investigated by changing the head of the hammer.
3) The data is reduced to modal information through modern modal testing procedures, thus reducing the measured behavior to the basic structural characteristics.
4) System identification techniques can then be used in estimating parameters to more closely match calculated and observed behavior and to determine damping parameters, and parameters representative of support interaction with the structure, for instance.
5) Based on a finite element model and these measurements, the effects of proposed structural modifications on the dynamic characteristics, and ultimately the load capacity, may be determined in a quantitative fashion.

There are many questions to be answered before such a procedure becomes routine.

1) How the frequencies, the fundamental as well as the higher ones, affected by degration, local member failures, foundation and support effects?
2) Will the damping at low levels of motion and the form of the mode shapes give us an indication of degradation of the structure?
3) Will simple dynamic tests be sufficient to pick out troublesome bridges in an easily implemented inspection format or will static tests be required as well?
4) Will this testing procedure be useful in developing knowledge in the repair, retrofit, rehabilitation, restoration renovation, and reconstruction of existing bridges?

This proposal addresses these questions, and based on measurements of existing bridges, we hope to develop, implement, and evaluate this methodology of bridge testing. The results of this study will be instrumental at arriving at a national program that is comprehensive, easily implemented, and reliable for structural evaluation of existing bridge structures.