

Vehicle/Bridge Interaction For Medium Span Bridges—Research Element 6 Of The OECD IR6 DIVINE Project

Sara Barella and Reto Cantieni

EMPA, Swiss Federal Laboratories for Materials Testing, Switzerland

ABSTRACT

Besides static deformations, highway bridges also exhibit dynamic responses to the passage of heavy vehicles. The first part of the report covers the experiences gained by EMPA through a large number of dynamic loading tests. The main result is that bridges with a fundamental natural frequency $f \approx 2.5\text{--}4$ Hz are most susceptible to the dynamic actions of vehicles with steel leaf suspensions. This is due to the fact that the leaves are locked under usual conditions and the wheel loads hence show a predominant frequency content in this same range. This effect is known as "frequency-matching".

Modern vehicles are most often equipped with low-friction or air suspensions. Such vehicles show predominant wheel load frequencies $f \approx 1.5\text{--}1.8$ Hz. The second part of this report describes the layout, execution and preliminary results of dynamic bridge loading tests performed in Switzerland in the context of the OECD DIVINE Research Project in 1994. The test vehicle was a five-axle tractor-semitrailer where the suspension system could be changed between steel leaf and air and which was instrumented to measure the dynamic wheel loads. The three bridges tested were selected such as to produce frequency-matching between their fundamental natural frequency and the wheel load frequencies associated with the respective suspension system. The test results confirmed the existence of frequency-matching effects. Vehicles with modern suspensions seem however not to be a severe danger for medium to long span highway bridges. The validity of this statement has to be checked for rough pavement conditions which were not covered by the tests.

1. INTRODUCTION

Planning of the tests performed in the context of the OECD DIVINE Project, Research Element 6, Bridge Testing, was based on two sources: a) the results of standard dynamic loading tests performed by EMPA on roughly 250 reinforced and prestressed concrete highway bridges in the last 70 years and b) the results of comprehensive tests performed on the Deibüel Bridge in 1978.

1.1 THE EMPA STANDARD TESTS

A 160 kN two-axle leaf sprung test vehicle is driven over a highway bridge in its center line. Vehicle speed ranges between $v = 5$ km/h and the technically possible maximum speed, typically $v = 50\text{--}80$ km/h. The usual speed increment is $\Delta v = 5$ km/h but may be smaller in the range where maximum bridge response occurs. Crawl-speed tests yield the quasi-static bridge response. Two sets of tests are performed, one with the vehicle being driven on the usual pavement and one with the vehicle crossing a 50 mm x 300 mm plank being placed in the middle of the largest bridge span.

From the bridge deflection signals measured in several points on the structure, the bridge natural frequency f and damping ζ are determined as well as the dynamic increment ϕ for the most significant point:

$$\phi = \frac{A_{\text{dyn}} - A_{\text{stat}}}{A_{\text{stat}}} \cdot 100 [\%] \quad (1)$$

where A_{stat} and A_{dyn} are the quasi-static and maximum dynamic bridge responses respectively.

Summarizing the results of roughly 250 tests performed on reinforced and prestressed concrete bridges yields the following [1]:

- The bridge total length is $L = 13\text{--}3,147$ m with a mean value $L = 156$ m.
- The maximum span length is $l = 11\text{--}119$ m with a mean value $l = 40$ m.
- Flexural stiffness (related to the maximum deflection) of the bridges is $k = 7\text{--}800$ kN/mm with a mean value $k = 173$ kN/mm.
- The bridge fundamental natural frequency is $f = 1.23\text{--}14$ Hz with a mean value $f = 3.66$ Hz.
- Damping of the bridge fundamental vibration is $\zeta = 0.3\text{--}5.7\%$ with a mean value $\zeta = 1.31\%$ ($\% =$ percentage of critical).
- The dynamic increment for passages without the plank is largest for bridges with a natural frequency $f = 2.5\text{--}4$ Hz. The maximum value is $\phi = 65\%$ (Fig. 1).

- For passages with a plank on the pavement, the bridges with $f = 1.8...3$ Hz and $f > 7$ Hz are most susceptible to the dynamic actions of the vehicle. The maximum value for both cases is $\phi = 230\%$ (Fig. 2).

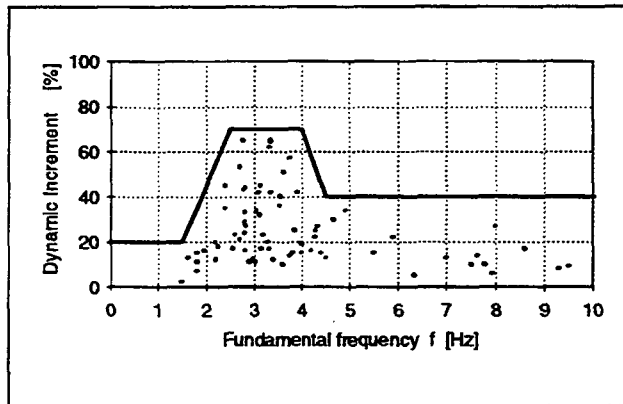


Fig. 1 Maximum dynamic increments for passages without the plank.

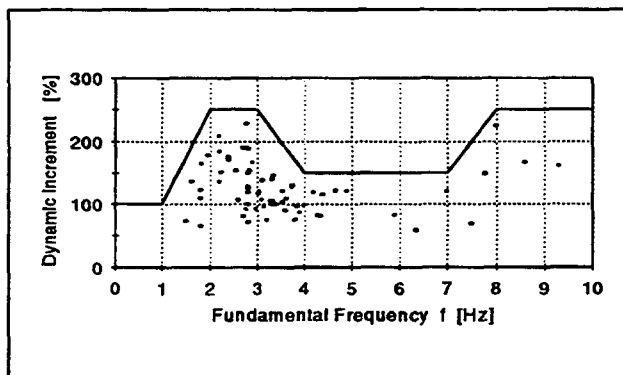


Fig. 2 Dynamic increments for passages with the plank.

1.2 THE TESTS ON THE DEIBÜEL BRIDGE

There were four goals concerning the dynamic bridge response to be reached with these tests: a) to determine the effect of the vehicle type, b) to determine the effect of the tire pressure, c) to determine the effect of pavement unevenness and d) to get insight into the laws governing the process through simultaneous measurement of the dynamic wheel loads and bridge response and subsequent analysis of the vehicle/bridge-interaction behavior.

The Deibüel Bridge is 110 m long with a maximum span of 41 m and has a fundamental natural frequency $f = 3.03$ Hz. This means that the bridge can be supposed to be very susceptible to the dynamic actions of heavy traffic. More details of this bridge are given later in this paper.

The pavement quality was rather poor and medium respectively. According to Swiss Standards [2], the poor pavement was close to the limits where it should have been rehabilitated. The characteristic values according to ISO/DIS 8608 [3] were:

- $G_d(\Omega_0) = 24 \cdot 10^{-6} \text{ m}^3$, Road Class B
- $G_d(\Omega_0) = 2.3 \cdot 10^{-6} \text{ m}^3$, Road Class C

The test vehicle fleet consisted of 14 different vehicles with 2...5 axles and a gross weight of 160...400 kN. Tests were mainly performed with one, in some cases with two and

four vehicles simultaneously driving over the bridge. All vehicles were equipped with steel leaf spring suspensions. The tire pressure was 6, 8 or 10 bar. The test results can be summarized as follows [4], [5]:

- The highest dynamic increments ($\phi = 75...85\%$) were measured for three- and four-axle rigid trucks on the rough pavement.
- The maximum dynamic increments for two-axle trucks reached values $\phi = 60...73\%$ for the rough and $\phi = 55...60\%$ for the smoother pavement. As expected, and confirming the results of the EMPA standard tests, the Deibüel Bridge is hence very susceptible to the dynamic actions of heavy vehicles.
- For combined vehicles the maximum dynamic increment was in the range $\phi = 25...50\%$ and for multiple presence in the range $\phi = 10...35\%$.
- It was not possible to derive any general rule concerning the dependency of the dynamic increment on the tire pressure or the pavement unevenness.
- Frequency analysis of the dynamic wheel loads showed that the dominant frequency is $f = 2.5...3$ Hz for speeds up to $v = 25$ km/h for the rough and up to $v = 50$ km/h for the smoother pavement. This means that the leaf spring suspensions are locked for speeds below these values. It can hence be assumed that for a good pavement quality the leaf springs are locked up to $v \approx 80$ km/h. The reason for the susceptibility of bridges with $f \approx 3$ Hz is hence that this is the range where the dynamic wheel loads of leaf sprung vehicles predominantly occur: "frequency matching" (Figs. 3 and 4).
- Investigation of the vehicle/bridge interaction behavior showed that this interaction varies between zero, when the vehicle is in a support region and very large, when the vehicle is in a mid-span region. Maximum interaction results in a shift of the dominant bridge vibration frequency of $\pm \Delta f = 0.25$ Hz which is roughly $\pm 10\%$ of the bridge fundamental frequency $f = 3.03$ Hz. This shift is due to close coupling of the dynamic systems "vehicle" and "bridge" whereby the vehicle mass amounted to 3.5% of the bridge modal mass.

As a result of the knowledge gained through the EMPA standard tests and the tests on the Deibüel Bridge the curves given in Figure 5 were proposed as a provision concerning dynamic load allowance for highway bridge loading in the new Swiss Code SIA 160 (1989), "Actions on Structures" [6]. This provision is only valid for pavements fulfilling the minimum surface evenness requirements according to [2].

1.3 LAYOUT OF THE OECD DIVINE BRIDGE TESTS

All of the tests described in the last two paragraphs were performed with using steel leaf sprung vehicles. Experience showed that the leafs do not unlock under usual conditions concerning pavement roughness and speed. As a result, the predominant wheel load frequency is $f = 2.5...3$ Hz instead of the $f \approx 1.5$ Hz for the suspension with unlocked leafs. Hence, bridges with a fundamental frequency $f = 2.5...3$ Hz, i.e. with a maximum span length

$l \approx 40$ m are most susceptible to the dynamic actions of heavy vehicles.

Modern vehicle suspensions are of the low-friction or the air spring type where damping is provided by shock absorbers. As no dry friction is to be surmounted, the dynamic wheel loads of such vehicles will be $f = 1.5 \dots 1.8$ Hz under all circumstances. The question to be answered was hence straightforward:

- Is there any risk, that bridges with a fundamental frequency $f = 1.5 \dots 1.8$ Hz, i.e. with a maximum span length $l = 60 \dots 70$ m, are susceptible to the dynamic actions of vehicles equipped with modern suspensions? If yes: What can the maximum dynamic increment be?

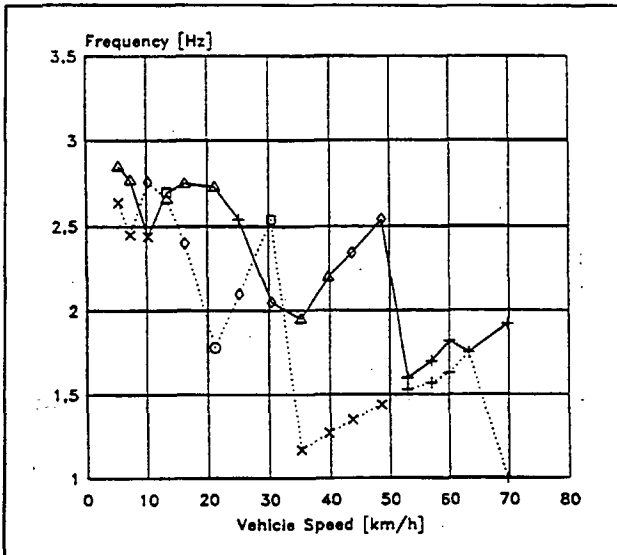


Fig. 3 Predominant wheel load frequencies for the rough pavement. Solid line: rear axle, dotted line: front axle. The symbols indicate the shape of vibration which is between pure heave, squares, and pure pitch, + crosses.

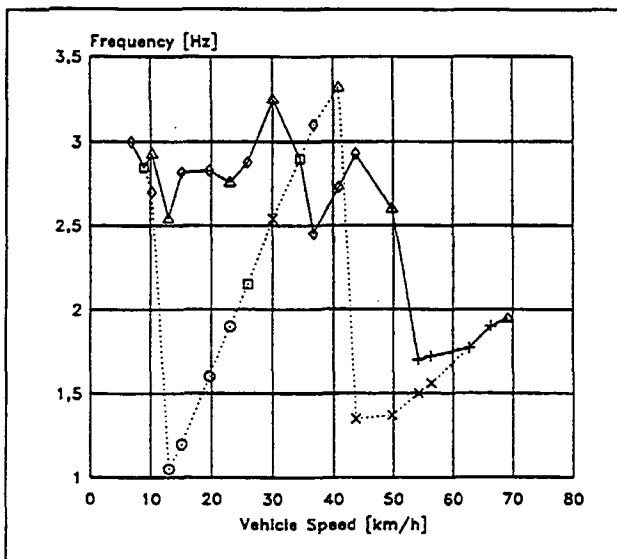


Fig. 4 Predominant wheel load frequencies for the smooth pavement. (Further comments: see caption of Figure 3).

With the NRC test vehicle (see description below) being available it was hence straightforward to choose two bridges with fundamental frequencies $f \approx 3$ Hz and $f \approx 1.6$ Hz and to perform tests with this vehicle equipped with steel and air suspensions respectively. For comparison, a third bridge with $f \approx 4.5$ Hz was also tested.

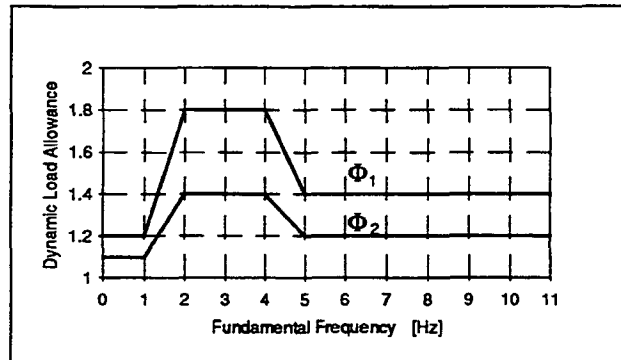


Fig. 5 Proposed dynamic load allowance for the new Swiss Loading Code (Φ_1 is for a tandem axle group, Φ_2 for a lane load).

2. THE OECD DIVINE BRIDGE TEST PARAMETERS

2.1 THE BRIDGES

The bridges tested in Switzerland in April/May 1994 were Föss, Deibüel and Sort, with natural fundamental frequencies $f = 4.44, 3.03$ and 1.62 Hz respectively. These all are prestressed concrete structures. The geometry of the bridges is indicated in the Figures 6 to 8. The instrumentation is indicated in Figure 8. All instruments were inductive displacement transducers to measure vertical deflection. In order to determine the effective vehicle speed, three contact thresholds were fixed to the pavement. One at the beginning of the test track and the other two at both ends of the bridge.

Föss bridge, located in the southern part of the highway crossing the Gotthard Pass has a total length $L = 79$ m and three spans $l = 24, 31$ and 24 m. The cross-section is a double box-girder of 1.59 m depth and 17.0 m width. The two field supports consist of two columns with a 2 m by 1 m cross section each. The superstructure is curved with a constant radius $R = 200$ m. The static system of the superstructure can be understood as a girder continuous over three spans with the fix point at one of the abutments.

Deibüel bridge, located on the National Highway N4a between Zurich and Lucerne is a straight bridge with $L = 110.3$ m and three spans $l \approx 37, 41$ and 32 m. Its one-cell box-girder is 1.80 m deep and 11.75 m wide. The cross section of the two piers is 3 m by 1 m. One of the piers is clamped into the box-girder, the connection between the other one and the girder is designed as a concrete hinge whereas the abutments are horizontally free. This makes the bridge behave as a frame.

Sort bridge leads the Cantonal Highway in the vicinity of Airolo, a village south of the Gotthard Pass, over the National Highway N2 Basel - Chiasso and the Ticino River. With a total length $L = 258.8$ m, the lengths of the five

spans are roughly $l = 36, 58, 70, 58$ and 36 m. The depth of the one-cell box girder varies between 2 m and 2.8 m, the bridge deck is 11 m wide. The four piers have a circular section with a 1.6 m diameter. The radius of curvature of the superstructure is $R = 900$ m. The superstructure, again a continuous beam, is connected to the substructure with roller bearings at the abutments and piers. The horizontal fix point is located at one of the piers.

The natural frequencies and associated mode shapes of the bridges were measured by means of specific tests. These so-called "Ambient Vibration" tests were performed where the usual traffic served as source of vibration [7], [8]. For the bridge Deibüel only the first six frequencies are given. However, through the tests a larger number of frequencies were found but still had not been analyzed by the time of writing this paper. The measured frequencies are given in Figure 9 and an example of the vertical mode shapes of Sort bridge is shown in Figure 10.

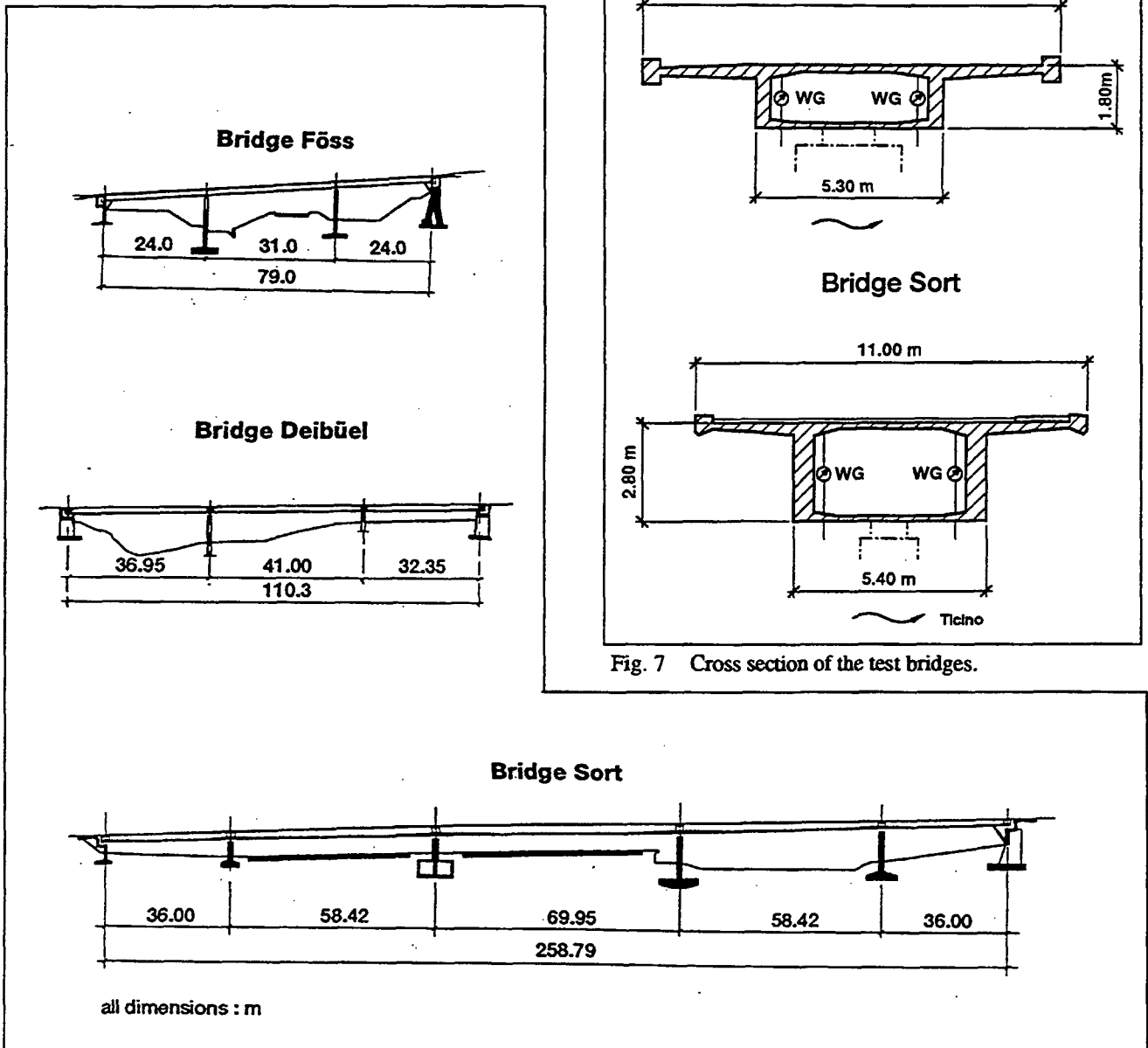


Fig. 7 Cross section of the test bridges.

Fig. 6 Longitudinal section of the test bridges.

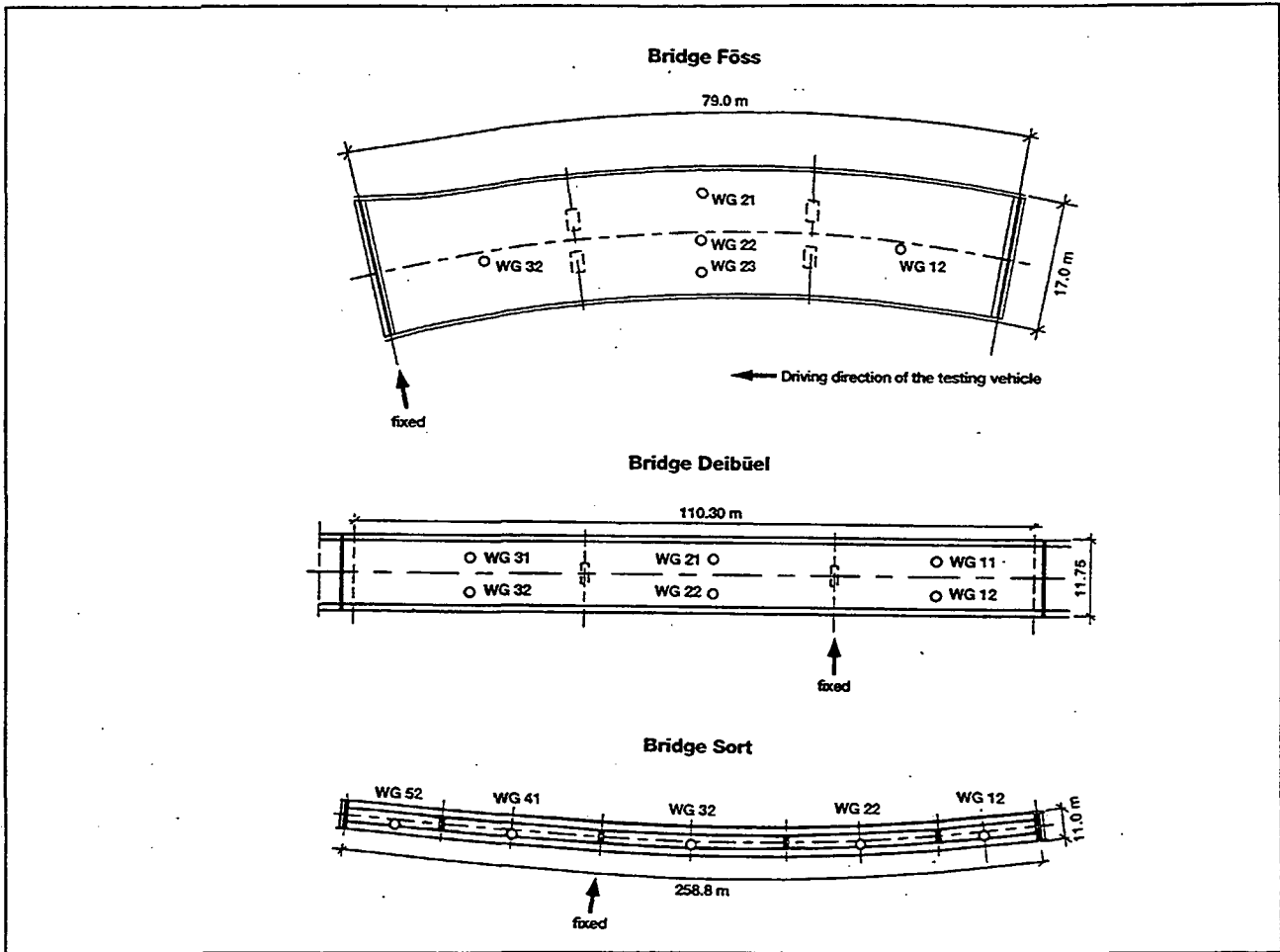


Fig. 8 Instrumentation of the test bridges (WG = inductive displacement transducer).

Mode Nr.	Frequency [Hz]					
	Föss		Deibüel		Sort	
1	4.03	H	3.00	B	1.04	H
2	4.44	B	3.80	H	1.62	B
3	6.45	B	4.25	B	1.73	H
4	7.10	T	5.45	B	2.45	B
5	7.76	H	6.70	T	2.55	H
6	---	---	7.50	T	2.85	T
7	---	---	---	---	2.98	B
8	---	---	---	---	4.04	H
9	---	---	---	---	4.31	T
10	---	---	---	---	4.51	B

B = longitudinal bending V = transverse bending
 T = torsion H = horizontal

Fig. 9 Natural frequencies of the test bridges.

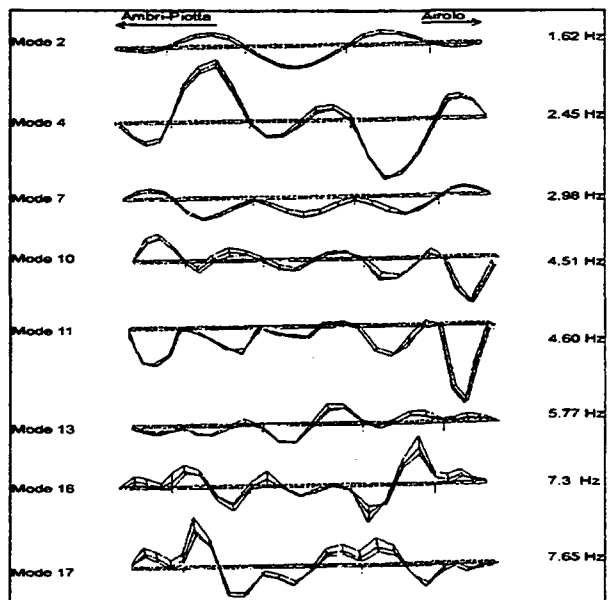


Fig. 10 Some mode shapes of the Sort bridge of the longitudinal bending type.

2.2 THE TEST VEHICLE

The test vehicle was a five-axle tractor-semitrailer owned by the National Research Council of Canada (NRC), Ottawa. Its geometry is given in Figure 11. For the tests only two of the trailer tank's four compartments were filled with water so that the vehicle gross weight was 450 kN. The individual axle loads are also indicated in Figure 11. The vehicle being too heavy, too long and too wide, special permits were necessary to operate it in Europe. The driver and the operators of the on-board electronic equipment were members of the NRC staff.

The NRC test vehicle offers two features which make it most probably unique in the world:

- Its suspension system can be exchanged between steel leaf and air suspension. This does not apply for the steer axle which is always equipped with a steel leaf suspension.

- All wheels are instrumented so that the dynamic wheel loads can be continuously measured.

The wheel load measurement instrumentation consists basically of shear gauges and accelerometers, both mounted on the axle close to the wheel hub. The method used to derive wheel loads from the corresponding signals is described in detail in [9].

In addition to the wheel load measurement instrumentation the vehicle is equipped with sophisticated devices to allow the driver to accurately control vehicle speed and to exactly identify the vehicle location on the test track. Both devices, operating with contactless optical sensors are activated through white, reflective strips glued on the pavement at distances of 30 m. At the beginning and end of the test track, four of these strips were glued at distances of 190 mm between each other to trigger the on-board data acquisition system.

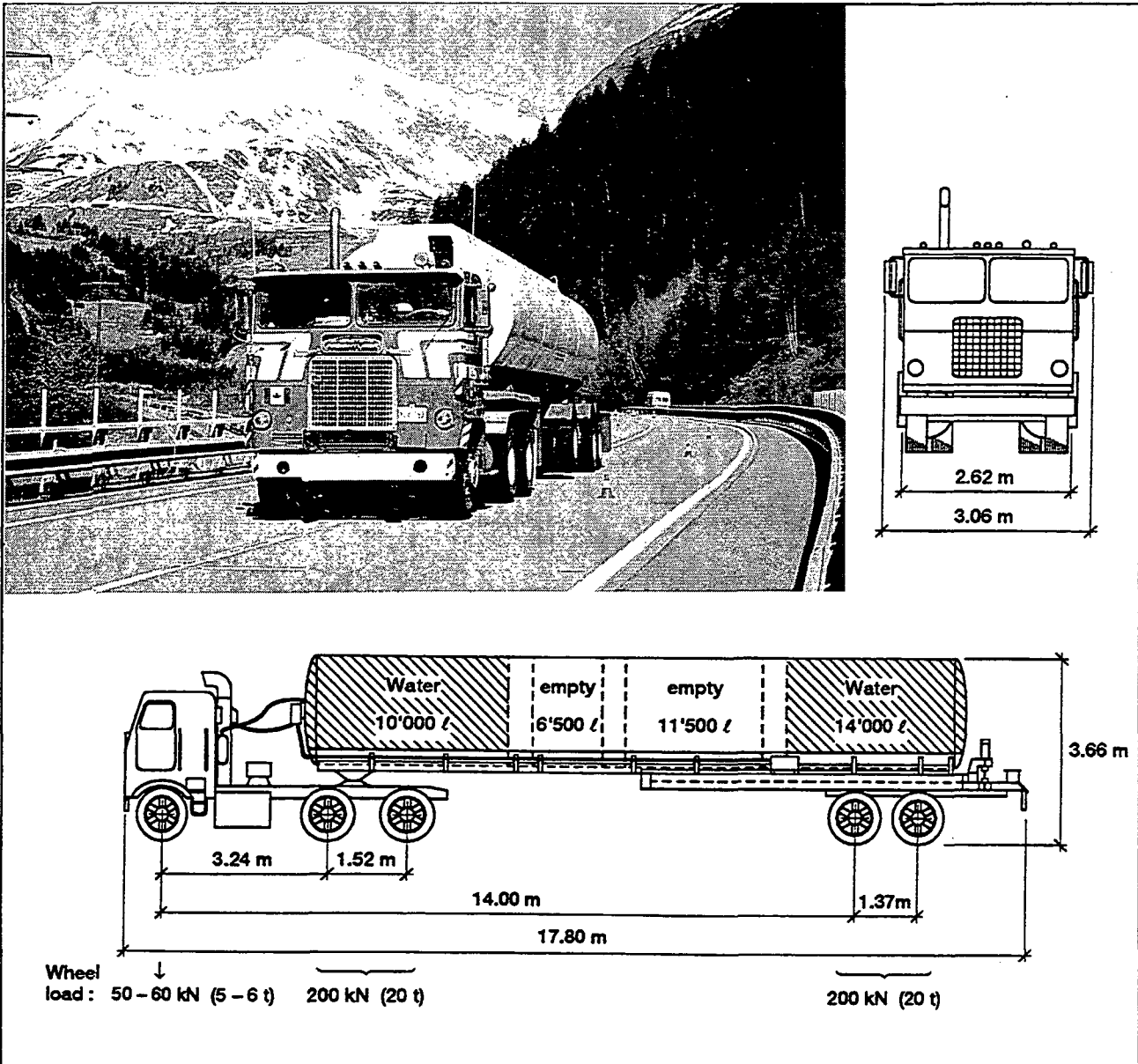


Fig. 11 The NRC test vehicle

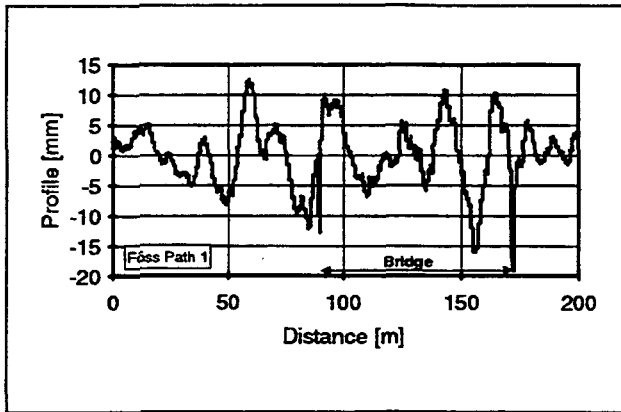


Fig. 12 Pavement profile of the Föss test track. It can be seen that the track consisted not only of the bridge but of a stretch before and after the bridge.

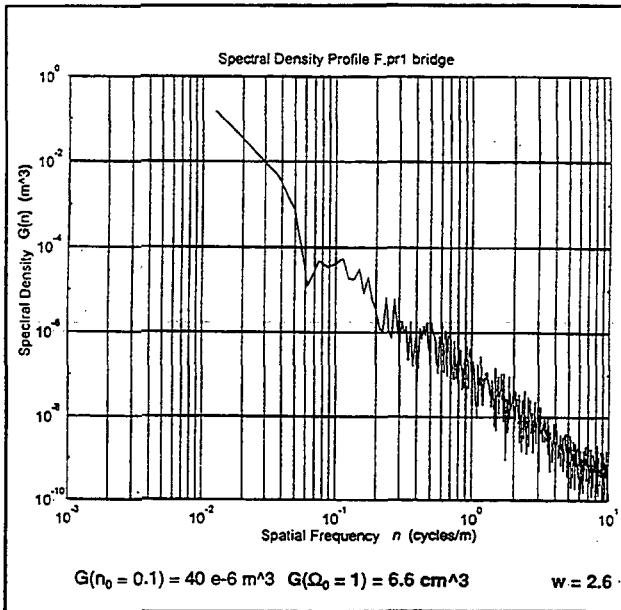


Fig. 13 Unevenness Power Spectral Density of the track given in Figure 12.

Bridge	Path	Gd (Ω) [$\times 10^{-6} \text{ m}^3$]	Waviness
Deibüel	1	1.8	2.20
	2	2.8	2.10
Föss	1	6.6	2.60
	2	6.6	2.60
Sort	1	1.7	2.00
	2	0.5	2.10

Fig. 14 Characteristic unevenness Power Spectral Density values of the pavements according to [3].

2.3 THE PAVEMENTS

The longitudinal pavement profile of the test tracks was measured in both wheel paths by Road Survey Technology Sweden AB (RST), with using the Laser RST Portable Profilometer being mounted on an EMPA car. This measurement system operates with two transducers mounted on a stiff bar fixed to the car's trailer hook. Each consists of a reflective laser sensor contactlessly measuring the distance between the supporting bar and pavement surface plus an accelerometer measuring the acceleration on top of the laser. This acceleration signal is then integrated twice to yield the absolute vertical movement of the laser sensor. The difference of the two signals is the longitudinal pavement profile. The necessary electronics and data acquisition equipment is traveling on-board the car. The measurements were performed at vehicle speeds of $v = 35 \text{ km/h}$ and $v = 70 \text{ km/h}$ respectively.

From the measured profile signals the unevenness power spectra were calculated at The Swedish Road and Traffic Research Institute, VTI.

Figures 12 and 13 give an example of the longitudinal profile of the Föss test track and the associated power spectral density diagram as determined by RST and VTI. The table in Figure 14 shows the characteristic values of pavement surface unevenness according to ISO [3] for each bridge and wheel path. The pavements were smooth for the Föss, smooth to very smooth for the Deibüel and very smooth for the Sort bridge.

3. TEST PROGRAM AND -PROCEDURES

3.1 THE TEST PROGRAM

Prior to the test vehicle's arrival in Switzerland, the three bridges were instrumented and the strips used for speed/position detection of the vehicle were stuck to the pavement. With the truck having arrived at EMPA, its instrumentation was checked and calibrated, the water tanks were filled and the static wheel loads were exactly measured.

The bridge tests were performed in three phases between mid of April and end of May 1994. In the first two weeks, the three bridges were tested with the NRC vehicle equipped with the steel leaf suspension. The suspension was then changed in a Swiss Army facility and in the last two weeks bridge testing was repeated with the air-sprung vehicle. The total number of test runs performed amounted to roughly 400.

Management of the normal traffic was not identical for the three bridges. The easiest case was Föss bridge, because the winter closure of the highway crossing the Gotthard pass was effective until the last day of testing. Föss bridge being a relatively short structure, testing could be accomplished in one day. This was of course only possible thanks to the brilliant backing-up capability of the NRC driver.

Deibüel bridge is actually a twin-structure where each of the well separated bridges carries two lanes of the four-lane N4a. Here, the traffic of the Zürich-Lucerne lanes had to be deviated and the lanes Lucerne-Zürich separated to serve one direction each. Undisturbed testing was possible between 9 am and 4 pm for three days in April and May.

At Sort bridge, the situation was rather difficult because it was not possible to close the highway completely for a longer period of time. Traffic management was accomplished using flagmen here.

Even though a two weeks period was reserved for changing of the suspension system, this work performed at the Swiss Army Motor Vehicle Facility (AMP) Hinwil took only five days. It can be mentioned here that perfect co-operation between the NRC, EMPA and AMP teams and the unlimited dedication of all persons involved in planning and execution of the project were the basis for the finally achieved success.

3.2 TEST PROCEDURES

The test procedures were the same as described in paragraph 1.1 for the EMPA standard tests. Three exceptions may be mentioned: a) no tests were performed with crossing a plank, b) the speed increment was kept as low as $\Delta v = 2$ km/h and c) the test vehicle tire pressure was measured at the beginning and at the end of every day of testing.

4. DATA ACQUISITION AND -PROCESSING

4.1 DATA ACQUISITION

Two data acquisition systems were operated on-site: a) The NRC MegaDac system plus computer on-board of the test vehicle and b) the EMPA PCM-system located in a stationary measurement van. The connection between the two was provided by an eight-channel 2.45 GHz radio telemetry link sending the EMPA signals to the NRC MegaDac system. On-line checking of the signals revealed that the telemetry link unfortunately failed after two thirds of the test program were accomplished. This will lead to some problems when studying the vehicle/bridge interaction behavior. The exact location of the NRC white strips and of the EMPA contact thresholds being known it should however be possible to cope with these problems.

Two sets of data were hence available after a test: The bridge plus contact threshold signals on the EMPA magnetic tape and all vehicle signals in digitized form on the NRC computer harddisk. In case the telemetry link had been working properly, the NRC harddisk data also contained the EMPA signals. The NRC data were copied to an EMPA computer every day.

4.2 TIME DOMAIN ANALYSIS

Two tasks had to be performed: a) determination of the bridge deflection dynamic increments and b) calculation of the wheel load signals from the measured strain and acceleration signals.

The dynamic increments were determined by playing back the signals stored on PCM-tape, transferring them into a computer and applying software packages developed by EMPA (see also Paragraph 1.1). It may be mentioned here that determination of the quasi-static comparative value A_{stat} is achieved through digitally low-pass filtering of the signals measured for at least three crawl-speed tests and subsequent averaging of the maxima.

To determine the wheel load signals, software packages developed at NRC were used. EMPA was again strongly supported by NRC here.

4.3 FREQUENCY DOMAIN ANALYSIS

In the contrary to the Deibüel tests described in Paragraph 1.2, the relevant time signals were not available in analog form. New processing procedures had hence to be developed to directly deal with the digitized signals. First attempts to calculate frequency spectra were undertaken with using software packages developed at NRC. However, in the meantime very efficient software packages, actually developed to process Ambient Vibration Tests as mentioned earlier, have been installed at EMPA [7], [8]. The author of these packages is staying at EMPA for one year and he is now also processing the signals of the bridge tests in the frequency domain. This covers calculation of the wheel load and bridge deflection frequency spectra as well as investigation of the vehicle/bridge interaction behavior through studying the cross power spectra of wheel load and bridge deflection signals.

5. TEST RESULTS

5.1. TIME DOMAIN

The dynamic increments as a function of vehicle speed have been determined for all measurement instruments and test parameter configurations. Figure 15 gives the corresponding results in graphic form for the deflection in the largest span for the three bridges.

For the Föss bridge, the dynamic increments reach smaller values than for the Deibüel and Sort bridges. The results for steel leaf and air suspension are comparable with often slightly larger values for the steel suspension. The maximum values are $\phi = 15\%$ for the steel and $\phi = 12\%$ for the air suspension.

For the Deibüel bridge, the dynamic increments are significantly higher for the steel suspension than for the air suspension. The maximum values are $\phi = 21\%$ and $\phi = 5\%$ respectively.

On the contrary, the dynamic increments for the Sort bridge are significantly higher for the air suspension than for the steel suspension for speeds $v > 40$ km/h. The maximum values are $\phi = 26\%$ and $\phi = 10\%$ respectively. For lower speeds, the dynamic increments are very similar and with $\phi \leq 5\%$ on a rather low level.

5.2. FREQUENCY DOMAIN

Signal processing in the frequency domain was in full progress by the time this paper was written. Figure 16 gives an example of a wheel load frequency spectrum.

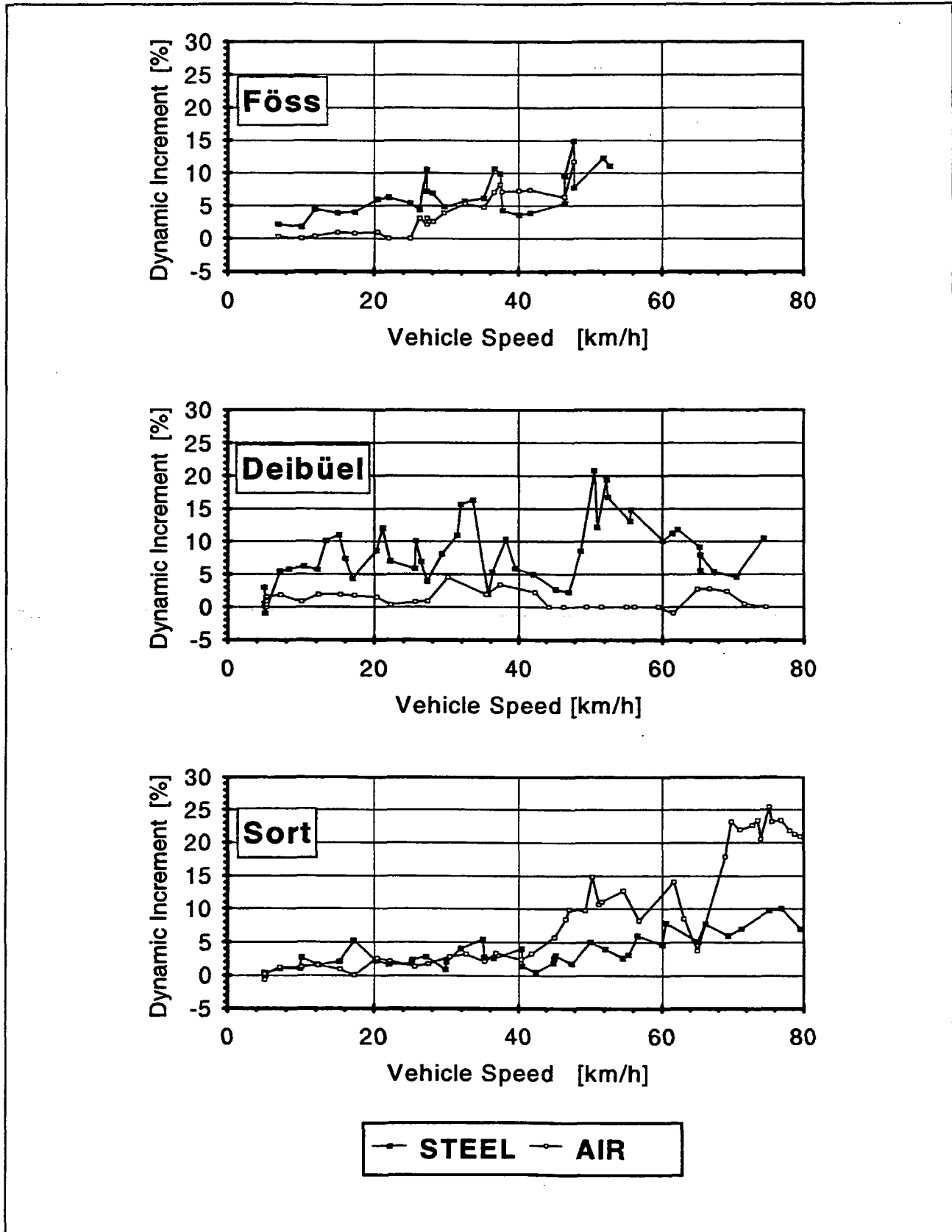


Fig. 15 Dynamic increments for the mid-span deflection of the largest span of the three bridges tested.

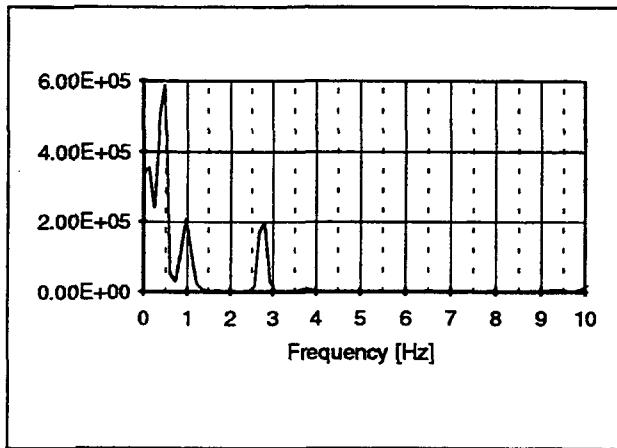


Fig. 16 Example of a wheel load Power Spectral Density Function. The peak at $f = 2.75$ Hz indicates that the leaf suspension was locked.

6. CONCLUSIONS

Comparison of the results for the two bridges of primary interest, Deibüel and Sort, yields that the expected quasi-resonance effects can actually be identified in the cases of frequency-matching. The Deibüel bridge with its fundamental frequency $f = 3.03$ Hz responds more strongly to the dynamic actions of the steel sprung vehicle, whereas the Sort bridge, $f = 1.6$ Hz, is more susceptible to the dynamic actions of the air sprung vehicle. From this point of view, the tests can be called a full success.

An additional comparison is possible with the results of the tests on the Deibüel bridge as discussed in Paragraph 1.2. The maximum dynamic increment for steel sprung articulated vehicles on the rough pavement was in the range $\phi = 25\text{...}40\%$ there. Hence, the code provisions given in Figure 5 seem to hold true for the case of leaf sprung vehicles under the restrictions concerning pavement unevenness according to [2].

The same conclusion can not be drawn from the OECD tests for the air sprung vehicle. Although the maximum values of the dynamic increment for the quasi-resonance cases Deibüel/steel and Sort/air are very close to each other, the case of rough pavement still has to be investigated for the Sort/air case. As it is not possible to change the pavement of the Sort bridge with reasonable effort, the problem will be studied with the help of computer simulation programs.

Several institutes all over the world will be involved in this last part of the OECD DIVINE Bridge Research Project. The various simulation programs will firstly be calibrated based on the test results. The crucial part here is the correct modeling of the vehicle suspension systems. Parameter studies with systematic variation of the pavement profiles will then be performed to also estimate an upper boundary for possible dynamic bridge response to the passage of air sprung vehicles.

7. ACKNOWLEDGMENTS

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