AMBIENT VIBRATION SURVEY OF THE FATIH SULTAN MEHMET (SECOND BOSPORUS) SUSPENSION BRIDGE

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SUMMARY

Ambient accelerations due to dynamic excitation by wind and traffic were measured on the deck, towers, cables and hangers of the Fatih suspension bridge. From these measurements it was possible to obtain natural frequencies, mode shapes and damping ratios for vertical, lateral, torsional and associated modes in the deck and tower up to a maximum of 2Hz.

The objective of the test was to validate mathematical modelling used in seismic analyses of the bridge. Agreement between experimental and theoretical modes was acceptable for vertical modes below 1Hz, and for torsional modes, but it was difficult to identify the lateral modes due to low levels of response.

The dynamic behaviour of this bridge and two other major European suspension bridges is discussed in relation to the differences in loading and structural design.

INTRODUCTION

This study is the second part of a programme of work on the Fatih Mehmet Sultan bridge which is the second suspension bridge across the Bosporus in Istanbul, Turkey, having a main span of 1090m. The first part of the work[1] comprised a comprehensive set of analyses of the response of the bridge to seismic excitation described as asynchronous or stochastic inputs.

The ambient vibration survey of Fatih bridge was the third in a series of prototype tests of major European box-girder suspension bridges by the Earthquake Engineering Research Centre (EERC) at Bristol University, Humber having been tested in 1985[2] and Bogazici (first Bosporus) in 1987[3]. These bridges were tested to validate the mathematical modelling used for seismic response analyses of these suspension bridges[1].

MATHEMATICAL MODELLING

Details of the bridge, the mathematical modelling and the predicted modes are given elsewhere[1] but the models used are summarised here, since the validation involves comparison of experimental and theoretical modes.

A three-dimensional model was used to determine the complete behaviour of the deck, cables and towers in the horizontal and vertical planes. Two-dimensional models were used to determine the behaviour of the bridge in the vertical and horizontal planes respectively. For each of these three models, two variants were used with different deck boundary conditions:

In variant SS the bearings at both ends of the deck are allowed to slide and hinge, by means of a hinged link mechanism at each end of the deck.

In variant FS the boundary conditions are changed so that one end of the deck is still hinged but is prevented from sliding.

OBJECTIVES

The objectives of the test were to determine experimentally the modal properties of the bridge in terms of its natural frequencies, mode shapes and damping ratios in order to validate the mathematical modelling described above and also to investigate the relationship between the loading and response amplitude. The intention was to determine vertical, lateral and torsional modes of the deck and longitudinal, lateral and torsional modes of the towers in the range 0-2Hz and to make measurements of hanger and main cable response and of wind conditions.

EQUIPMENT

The equipment used was similar to that used for the previous tests[2,3], with some additions, and is summarised in fig. 1.

The accelerations were sensed by up to five servo-type accelerometers and amplified by power supply/conditioner units; two of these accelerometers are capable of resolving acceleration signals as low as 10^{-6}g. Signals were recorded on a four channel FM tape recorder and some of the signals were processed on site in real time using a spectrum analyser to determine the Fourier spectra of the acceleration signals and to compute the amplitudes and phases of the signals. These spectra together with spectra obtained from the recorded signals which were not analysed on site were used to determine the natural frequencies and damping ratios of the Bridge and to determine the mode shapes from relative phases and amplitudes.

A cup and vane weather monitor was used to provide wind speed and direction data and a micro-computer was used to store Fourier spectra from the analyser and to record the wind data. As an exercise and a complement to the analog measurements, digital data acquisition equipment consisting of anti-alias filters and an analogue to digital converter was used to record acceleration signals on the computer during some of the measurements.

TEST PROGRAM

The Fatih Bridge was tested between the 6th and 15th June 1989 using a methodology similar to that used for the previous tests, on Humber[2] and Bogazici[3].

A reference accelerometer was set up at a strategic point inside the deck and some of the other ('traveller') accelerometers were placed at positions throughout the deck. The accelerometers were in turn oriented to measure lateral and vertical acceleration response, and by moving the travellers from one end of the deck to the other and up and down a tower, values for natural frequencies of the deck and towers were determined and the mode shapes mapped out. The 28 measurement positions were arranged as shown in fig. 1 (inset) with positions at every second hanger location in the European half span from mid-span to tower connection to obtain fine resolution of the mode shapes, at every fourth hanger location in the Asian half span to confirm the symmetry of the mode shapes, and at five levels in the European tower (foundation/bearing level, mid portal, upper portal and intermediate positions). The position (t1) in the upper portal was also used as a secondary reference for measurements of response of the tower while position 10 was used as reference for measurements in the deck.

Measurements were made of torsional response by placing accelerometers at opposite sides of the deck and at the top of the tower. By taking the sum and difference of the signals the vertical component of the vibrations could be separated from the torsional component.

Measurements were made of the longitudinal motion of the deck to check the performance of the deck bearings, of the vertical and lateral motion of the cable to check for the prolific cable modes predicted by the finite element analysis, and of vibrations in the hangers to estimate the tensions.

DATA ANALYSIS AND PARAMETER ESTIMATION

The data processing techniques applied at Fatih have been used successfully in numerous prototype testing activities and are well documented[2].

Power spectra of accelerations from individual measurements were studied to determine sets of values for frequency, damping ratio and amplitude. These three parameters were derived from the relatively simple procedure of fitting the best curve (in the sense of minimal least squares difference) representing the response of a single degree of freedom (SDOF) oscillator to the data in the region of a resonance.

Mode shapes were mapped out by normalising the modal amplitude values to the reference value (amplitude 1, phase 0) and assigning the in-phase value as the modal amplitude at the measurement position.

There are certainly errors intrinsic to this method, such as the need to assume a random input with a flat power spectrum, and the effects of nonstationarity, windowing and limited resolution. Nevertheless for this sort of data few of the known parameter estimation techniques can be used and of these, SDOF curve fitting is appears to be the simplest to use. Some studies of the bias and variance errors in the parameter estimates, as functions of frequency resolution and length of data record, have been made[4] so that the authors have some confidence in this method.

Where possible (for all vertical modes and higher lateral modes) frequencies and damping values have been determined from power spectra obtained by analysing response signals over a 60 hour period with a resolution of 0.002Hz. With these parameters, assuming stationarity, the overestimation[4] is equivalent to a contribution of at least 0.4% to the measured damping for a mode at 0.125Hz (e.g. $<0.93\% + <0.4\% = 1.33\%$), and this decreases roughly as the inverse of frequency. The variance errors (normalised as $\varepsilon = \sigma/\mu$) are estimated at 2% (of mean) for damping and $<0.1\%$ for frequency.

Values for other modes have been taken from 12-hour records and except for the lowest lateral modes the higher frequencies reduce the error bounds in the parameter estimation. The effects of nonstationarity have not been quantified.

PRESENTATION OF RESULTS

For brevity, only modes up to 1Hz are discussed in detail; the full set of modes is given in ref. 5.

Vertical modes

A check on the auto power spectrum of vertical acceleration of the deck at station 10 (the reference) in the range 0-6Hz, showed strong response between 2Hz and 4Hz indicative of dynamic loading due to vibration of vehicle bodies on their suspension systems and weaker response below 1Hz, mainly wind-induced. Fig. 2 is the auto power spectrum of vertical and torsional acceleration response in the range 0-1.6Hz, from the sum and difference of accelerometer signals either side of the deck. Vertical modes (fig. 2a) were in general very well defined and easy to identify.

Table 1 summarises the 12 vertical deck modes in the range 0-1Hz obtained from the vertical deck accelerations and fig. 3 shows matching of vertical experimental and theoretical mode shapes obtained from variant FS of the three-dimensional model.

The experimental vertical modes of vibration matched the theoretical modes obtained from the FS variant. The difference between the experimental and theoretical frequencies, indicated as percentage error, increases with frequency, reaching 6% (overestimate) for mode V12; the error accelerates for higher modes. When compared to the frequencies from the two-dimensional solution (which are slightly higher than those from the three-dimensional model) the overestimation is increased.

Torsional deck modes

Fig. 2b is the auto power spectrum of torsional response at station 10 (reference), Table 2 summarises the torsional modes (labelled T) and fig. 4 shows the experimental modes compared with the theoretical modes.

Note that while experimental mode T3 (0.53Hz) is clearly torsional the experimental and theoretical mode shapes do not match well at the centre span, which is surprising since it would be expected to fit into the ordered sequence of increasing numbers of nodes and antinodes.

Apart from the lowest mode, all the theoretical torsional modes underestimate the frequency by the same percentage (about 4.4%).

Lateral deck modes

Fig. 5 is the auto power spectrum of the signal from accelerometers placed to measure lateral acceleration response at station 22. Because the accelerometers cannot differentiate between lateral acceleration and the component of gravity due to deck rotation, the measured signal is mostly due to the summation of the dynamic lateral response and the quasi-static torsional response due to asymmetric traffic loads and wind forces. Even a modest RMS deck rotation of 0.01° adds 17×10^{-5}g to the low frequency part of the signal and the result is seen in Fig. 5. Short of making direct measurements of deck displacement[6] there is no practical way of extracting the weak dynamic lateral signal, which apart from a strong response around 0.3Hz is at least an order of magnitude less than for vertical motion in this bridge and lateral motion in Humber[2] and Bogazici[3].

Examination of spectra such as fig. 5 led to the identification of 8 possible lateral modes up to 1Hz and these are summarised in Table 3. The lowest clearly identifiable mode L1 is 0.077Hz, corresponding to a theoretical frequency of 0.073Hz. In the range 0.2-0.27Hz the acceleration spectra appear to show several modes; modes L2 and L3 appear in most of the individual measurements. The strong response at 0.297Hz is almost certainly due to mechanical cross-coupling of the accelerometers in the first torsional mode, but mode L4 appears to be a distinct lateral mode. Only one mode (L5 at 0.315Hz) can be identified clearly between 0.3Hz and 0.4Hz.

Fig. 6 shows the experimentally determined shapes for these modes. The circles map out the mode shapes and have diameters proportional to the value of transfer function coherence between the reference and the appropriate travelling accelerometer. The modes are so weak that even with the most sensitive accelerometers the signal to noise ratio (related to coherence) is low. The shapes do not follow the same sort of progression as the vertical modes and the correspondence with theoretical modes is not clear except for modes L1 and L2. The other theoretical modes indicated in Table 3 appear to match with experimental mode shapes and frequencies although the differences between measured and predicted frequencies vary in sign and amplitude. Some theoretical modes were not obtained experimentally and vice versa.

The damping values in Table 3 for modes L1-3 were obtained from spectra with very clear peaks obtained with relatively few averages because the spectra from 610 averages were not clear.

Longitudinal tower modes

Fig. 7 is the auto power spectrum of longitudinal response at the top of the European tower. By using one accelerometer in each pylon it was possible to take the sum of the signals as the pure longitudinal response (fig. 7a) and the difference as the pure torsional response (fig. 7b).

Apart from participation in some of the lower vertical deck modes, the strongest tower modes occur above 1Hz and all but three of the measured modes above 1Hz correspond to a deck mode. Fig. 8 shows longitudinal tower modes corresponding to some of the strongest peaks in spectra above 1Hz. The mode shapes, for pure longitudinal and torsional mode, are similar and reflect the stiffness of the back stay cables. By contrast the participation of the towers in most of the lowest vertical deck modes, up to 0.3Hz is characterised by a cantilever type of mode shape. For the pure longitudinal response these mode shapes are in agreement with the tower modes predicted by the two-dimensional model, but no comparisons can be made for the torsional modes above 1Hz.

Lateral tower modes

Fig. 9 is the auto power spectrum of lateral acceleration at the top of the west pylon of the European tower and Table 4 summarises the identifiable modes up to 1Hz. The acceleration levels are higher than for either lateral deck response or longitudinal tower response and the modes are clearly identifiable and unambiguous. Fig. 10 shows examples of the experimental mode shapes, which progress from a simple cantilever mode shape towards the type of mode shape found with the longitudinal tower modes. A high proportion of measured tower modes do not involve deck participation.

Cable modes

An accelerometer was placed on the lowest part of the main cable, at centre span (station 16), and the response was compared to that from an accelerometer at the reference station in the deck, in order to investigate the participation of the cable in deck modes and to identify modes in which the cable participates but the deck does not. The interest in cable modes, particularly in the lateral plane, arises from the multiplicity of cable-only modes in the lateral direction of the two-and three-dimensional mathematical models[1].

Fig. 11a is the main cable vertical acceleration auto power spectrum. It is no surprise that the peaks all coincide with vertical or torsional deck modes.

Fig. 11b shows the lateral cable acceleration auto power spectrum and Table 5 summarises this data up to 1Hz. The large number of lateral cable modes found is consistent with the high proportion of cable modes in the two- and three-dimensional mathematical models.

Probable or possible correspondence with deck lateral (L) modes, as judged by high values of coherence with deck acceleration at lateral deck natural frequencies is indicated by '*'. The mathematical modelling predicts increased lateral tower motion at higher frequencies for more complex cable mode shapes and although no simultaneous measurements were made of response in the tower and on the cable, possible correspondence with measured lateral tower (TL) modes is made, based on coincident frequency values. Some modes (e.g. CL1) do not correspond with any measured or theoretical lateral deck or tower mode.

Without having data from measurements in the towers, the deck and several points in the main cable it is not possible to be certain about the nature of all the cable modes.

Longitudinal deck response

Some of the measurements involved recording longitudinal response in the deck at the reference position and at the ends of the deck. The longitudinal measurements in the deck at the reference show peaks only at frequencies of vertical or torsional deck modes up to 1Hz; there are no pure longitudinal modes in this range. For this bridge the tower has zero response at the foundation level where the deck bearings are attached, so it was not possible to observe the bearing behaviour from coherence data for simultaneous measurements in the deck and tower at this level, as was possible for Bogazici and Humber.

Hanger response

Fig. 12 is the auto power spectrum of longitudinal acceleration of a hanger close to the reference. The strong modes shown occur at frequencies of

6.915Hz 10.436Hz 14.059Hz 17.744Hz

By contrast with measurements on the Bogazici Bridge[3], there is no strong mode at what would be expected as the fundamental frequency (3.45Hz) and the peaks are clear and sharp, despite RMS averaging over several records during which traffic loads (hence cable tension) would be expected to vary. Since hanger frequency varies with tension, this suggests modest variations in the hanger stress levels at Fatih by comparison to Bogazici. At the time of the respective measurements Bogazici was experiencing traffic loads of up to 140,000 vehicles per day, Fatih less than 24,000 vehicles per day.

Digital recordings of wind and response data

During the later part of the test a number of digital recordings were made of wind speed and direction concurrent with acceleration response.

The aim of these measurements was to identify separate (or continuous) periods during which wind conditions varied to the minimum degree so that the parameter estimation methods could use collections of data conforming to some definition of stationarity. Difficulties in using the equipment prevented it being used for this purpose and hence a secondary benefit of using the equipment was as a practice for future applications. The measurements that were made showed that except for a short period of strong gusts during which increased response levels were observed for the lower modes of vibration, the winds were generally fairly low, with maximum instantaneous values of about 8m/second, with the wind along the Bosporus, perpendicular to the deck.

COMPARISON WITH RESULTS FOR OTHER SUSPENSION BRIDGES

Although there have been numerous tests on long span suspension bridges, mostly in North America[7-10] and Japan[11,12], almost all of these bridges have been of the traditional truss-girder type. As this test was the last in a series of measurements on the alternative box-girder type, it is now possible to review the dynamic behaviour of this type of bridge. Table 6 therefore summarises the main features of the dynamic response characteristics of the Humber, Bogazici and Fatih bridges determined from prototype testing by EERC Bristol.

The character of the loading and hence response is different for each bridge, particularly for vertical vibrations. The lowest vertical modes are strongly dependent on wind excitation and the winds during the Humber test were strong and variable enough to prove this[2]. The effect of traffic can be seen clearly in the acceleration spectrum above 2Hz but there is also a component of this excitation upwards of almost 0Hz[13] and this leads to the enhanced response for Bogazici. It is almost certain that either the quasi-static (moving load) or dynamic excitation causes the different deck bearing behaviour at Fatih.

The pattern of estimated damping values for vertical modes is also different for each bridge, but in each case the figure of 2.5% used in the mathematical analyses is really an upper bound on measured values. Certainly the damping at Humber has a very strong aerodynamic component[14] and signal processing effects can contribute to overestimation at the lowest frequencies but other explanations need to be found for Fatih and Bogazici. The winds along the Bosporus were modest during each test so the aerodynamic influence cannot be great, and by a simple comparison, the values obtained at Fatih are consistently lower than Bogazici. The tempting explanation for this is the use of vertical rather than inclined hangers at Fatih but the larger live loads and response at Bogazici could be a stronger influence. The origin and measurement of damping in this type of suspension bridge is the subject of a continuing investigation.

For each Bridge the mathematical model predicts an antisymmetric mode whose frequency depends on which variant (FS or SS) is used in the model; variant FS, with the increased restraint has the higher frequency. For Humber and Bogazici the first two possible anti-symmetric modes have frequencies above and below that for the first symmetric mode; for Fatih they are both lower. Of the three bridges, only Bogazici has (weak) measured response in the SS variant anti-symmetric mode.

The lateral vibrations in Fatih are at least an order of magnitude less than the other two bridges and this is certainly a function of the lateral rigidity and aerodynamic resistance.

The different tower heights and arrangement of side spans leads to different longitudinal (and to a lesser extent lateral) vibration of the towers. The longitudinal modes are very weak in Fatih because of the back-stay restraint and the reduced height. In both Bosporus bridges, except for modes where the tower is driven by a vertical main cable vibration the mode shapes have a minimum value at the top of the tower.

For Humber and Bogazici the vertical and lateral modes agreed well with the measured modes, while for Fatih there is limited agreement for lateral modes and diverging values of frequency for vertical modes. The comparisons referred to in Table 6 are made with the two-dimensional models; frequencies for three-dimensional theoretical modes are lower, reducing the overestimation.

Torsional modes have been predicted with variable success. The pattern of mode shapes is predicted correctly, but the first mode is underestimated by about 15% in each case, with better agreement for higher modes.

CONCLUSIONS

1) The measured vertical natural frequencies and mode shapes match the predicted values for a mathematical model featuring bearings fixed at one end and free to slide at the other, rather than bearings free to slide at both ends. While there is almost exact agreement between the lowest experimental and theoretical vertical natural frequencies, there is an increasing divergence for increasing frequencies.

2) The vertical modes of the deck are clearly defined, while the lateral modes are weak, sparse and poorly defined. The extreme width of the deck (the widest box girder deck yet tested) is the probable cause.

3) The longitudinal tower modes clearly show the effect of the cable restraint as they have maximum amplitudes between the tips and the deck level and relatively high natural frequencies.

4) The lateral tower modes are stronger than the deck modes and have amplitudes comparable to those of the longitudinal tower modes.

5) The traffic loading for the bridge was low at the time of the test since only four of the eight carriageways were open. Even so, traffic was the main source of dynamic loading.

6) The hanger measurements suggest that the stress fluctuations in the hangers are small.

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REFERENCES

1 A. A. Dumanoglu, J. M. W. Brownjohn, R. T. Severn, 'Seismic Analysis of the Fatih Sultan Mehmet (Second Bosporus) Suspension Bridge', *Earthquake eng. struct. dyn.* ibid.

2 J. M. W. Brownjohn, A. A. Dumanoglu, R. T. Severn, C. A. Taylor, 'Ambient Vibration Measurements of the Humber Suspension Bridge and Comparison with Calculated Characteristics', *Proc. ICE, Pt. 2*, **83** 561-600 (1987).

3 J. M. W. Brownjohn, A. A. Dumanoglu, R. T. Severn, A. Blakeborough, 'Ambient Vibration Survey of the Bosporus Suspension Bridge', *Earthquake eng. struct. dyn.* **18**, 263-283 (1989).

4 J. M. W. Brownjohn 'Assessment of Structural Integrity by Dynamic Measurements' *PhD thesis*, Department of Civil Engineering, University of Bristol, 1988.

5 J. M. W. Brownjohn, A. A. Dumanoglu, R. T. Severn, 'Fatih Bridge Part II: Ambient Vibration Survey' *Report UBCE-EE-90-12*, University of Bristol, Department of Civil Engineering, 1990.

6 G. A. Stephen, E. Dagless, C. A. Taylor, 'A Visual Tracking System for the Measurement of Dynamic Structural Displacements.' *Concurrency: Practice and Experience*, **3** no. 4, 357-366 (1991).

7 A. M. Abdel-Ghaffar, R. H. Scanlan, 'Ambient Vibration Studies of Golden Gate Bridge: 1. Suspended Structure'. *J. eng. mech. div.* ASCE **111**, 463-482 (1985).

8 P. G. Buckland, R. Hooley, B. P. Morgenstern, J. H. Rainer, A. Van. Selst, 'Suspension Bridge Vibrations: Computed and Measured.' *J. struct. div.* ASCE **105**, 859-874 (1979).

9 V. R. Maclamore, G. C. Hart, I. R. Stubbs, 'Ambient Vibration Response of Two Suspension Bridges.' *J. struct. div.* ASCE **97**, 2567-2582 (1971).

10 T. Kumarasena, R. H. Scanlan, G. R. Morris, 'Deer Isle Bridge: Field and Computed Vibrations.' *J. struct. div.* ASCE **115**, 2313-2328 (1989).

11 I. Okauchi, A. Tanaka, K. Iwaya, N. Furuya, 'Vibration Test of Ohnaruto Test to Confirm Wind-proofness' *IABSE Symposium: Safety and Quality Assurance of Civil Engineering Structures,* Tokyo, 4-6 September 1986, 52-55.

12 K. Yamada, N. Yamamoto, H. Akiyama, 'Full-scale Dynamic Testing of Long-span Bridges.' Proc. 4th US-Japan workshop bridge eng. San Diego CA (1988).

13 J. M. W. Brownjohn, 'Humber Bridge Monitoring: Digital Measurements February-June 1991' *Report UBCE-EE-91-3*, University of Bristol, Department of Civil Engineering, 1991.

14 G. Diana, F. Cheli, A. Zasso, A. Collina, J. Brownjohn, 'Suspension Bridge Parameter Identification in Full Scale Test' 8th International Conference on Wind Engineering, London, Ontario, July 8-12 1991.

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Table 1 Vertical deck modes

Mode		Experimental frequency (Exp)/Hz	Theoretical frequency (Th)/Hz		%error (Th-Exp)/Exp		Damping %
no.	type and symmetry		3-d model	2-d model	3-d	2d	
V1	V asym	0.125	0.125	0.125	0.5	0.4	1.33
V2	V sym	0.155	0.159	0.159	2.8	2.9	1.27
V3	V sym	0.208	0.211	0.212	1.5	1.6	0.80
V4	V asym	0.244	0.250	0.251	2.4	2.7	0.69
V5	V sym	0.317	0.323	0.325	1.9	2.5	0.44
V7	V asym	0.389	0.396	0.400	1.9	2.7	0.38
V8	V sym	0.470	0.479	0.485	1.9	3.1	0.65
V9	V asym	0.555	0.568	0.576	2.4	3.8	0.29
V10	V sym	0.645	0.666	0.673	3.1	4.3	0.31
V11	V asym	0.741	0.772	0.787	4.2	6.3	0.63
V12	V sym	0.839	0.887	0.907	5.7	8.1	0.28
V13	V asym	0.942	-	1.037		10.0	0.28

Table 2 Torsional deck modes

Mode	Type and symmetry	Experimental frequency (Exp)/Hz	Theoretical frequency (Th)/Hz	% Error (Th-Exp)/Exp	Damping %
T1	T sym	0.296	0.243	-20.	0.70
T2	T asym	0.352	0.333	-4.5	0.77
T3	T sym?	0.529	0.501	-4.4	1.37
T4	T asym	0.692	0.661	-4.5	0.64
T5	T sym	0.867	0.828	-4.5	0.54

Table 3 Lateral deck modes

Mode	Experimental frequency /Hz	Theoretical frequency /Hz (2-d model)	Damping %
L1	0.077	0.073	<14.47
L2	0.239	0.218	<3.04
L3	0.250		<2.37
L4	0.287	0.288	0.97
L5	0.315	0.303	3.51
L6	0.432	0.421	1.39
L7	0.466		0.90
L8	0.504	0.543	0.78

Table 4 **Lateral tower modes**

Mode	Participation	Experimental frequency /Hz	Damping %
TL1	tower + deck	0.287	1.40
TL2	tower + deck	0.295	1.07
TL3	tower + deck	0.432	1.19
TL4	tower + deck	0.464	0.65
TL5	tower + deck	0.503	0.97
TL6	tower	0.520	1.03
TL7	tower	0.630	0.98
TL8	tower	0.673	0.15
TL9	tower	0.691	0.30
TL10	tower	0.753	0.24
TL11	tower	0.802	0.20
TL12	tower + deck	0.866	0.07
TL13	tower	0.937	1.03

Table 5 Lateral cable modes

Mode	Deck/tower mode no. and type	Experimental frequency /Hz	Damping%
CL1		0.155	1.59
CL2		0.221	3.12
CL3	L2	0.236	2.85
CL4	L3	0.250	1.71
CL5		0.266	1.90
CL6	L4*	0.286	0.57
CL7	L5*	0.296	0.92
CL8	L6	0.317	0.63
CL9	L7*	0.430	1.30
CL10	L8	0.470	0.36
CL11	L9	0.505	0.74
CL12	TL6	0.527	0.48
CL13		0.582	1.49
CL14	TL7?	0.623	1.47
CL15	TL7?	0.636	1.14
CL16		0.644	0.54
CL17	TL10	0.754	0.12
CL18		0.813	0.22
CL19		0.838	0.30
CL20	L10	0.865	0.14
CL21	TL13	0.941	0.27

* relatively strong coherence with lateral deck response at these frequencies

Observation	Fatih	Bogazici	Humber
Character of vertical loading and response	Light breeze, traffic ≈ 20,000 vehicles/day. Clear response for all modes.	High traffic load ≈150,000 vehicles per day, minimal wind. Weak response below 0.2Hz.	Traffic ≈10,000 vehicles per day, wind speeds can be high with strong effect on response.
Damping ratios for vertical modes	Apart from first 3 or 4 modes, values are mostly between 0.3% and 0.7%; no trend.	First 2 modes high, thereafter trend is to increase with frequency in range 0.5%-1.5%.	Values for lowest modes highly dependent on wind speed; trend to decrease with frequency.
Nature of experimental fundamental vertical modes	1st mode is antisymmetric and has higher frequency predicted by FS variant.	Weak response for 1st antisymmetric modes from *both* variants at frequencies above and below strong 1st symmetric mode.	1st antisymmetric mode at frequency predicted by FS variant, above first symmetric mode.
Strength of lateral modes	Very weak; $\sim 10^{-5}$g.	Generally fairly clear, $\sim 10^{-4}$g at tip; few modes from 0.5-1.0Hz.	Generally fairly clear, $\sim 10^{-4}$g; little response above 0.8Hz
Longitudinal tower modes: frequencies	Deck modes < 1Hz represented by weak response $< 10^{-5}$g at tip; stronger above 1Hz.	Response $\sim 10^{-4}$g at tip with deck modes well represented and clear.	Response mostly below $\sim 10^{-5}$g, deck modes present in both towers, long side span gives Barton lower frequency pure tower modes.
Longitudinal tower modes: mode shapes	Tower participation in deck modes is as simple cantilever, others show constraint at tower tip.	Tower participation in deck modes is as simple cantilever, others show constraint at tower tip.	Progression of cantilever modes from 0 to 1 or 2 nodes.
Accuracy of model: vertical modes	Overestimation error increases to 24% for experimental mode at 1.49Hz (12% 1.05Hz)	Overestimation error increases from 3% (for mode 1) to 5% for mode at 1.01Hz.	Overestimation error less than 2% below 1Hz, increases to 5% for mode at 1.5Hz
Accuracy of model: lateral modes	Mode 1 close; only two clearly corresponding modes identified (very weak response).	Good agreement for nine modes up to 0.76Hz	Agreement for 16 modes up to 0.78Hz, frequencies underestimated.
Accuracy of model: torsional modes	Mode 1 predicted too low (-18%), others about -5%. Modes weak but 1:1 matching possible.	Mode 1 predicted too low (-14%), others closer. Experimental mode 2 has two close frequencies.	Only two in first 40 modes. Mode 1 predicted too low (-15%), mode 2 close, but is weakly duplicated.

accelerations are RMS values

Table 6 Comparison of experimental and theoretical results from 3 box girder bridges

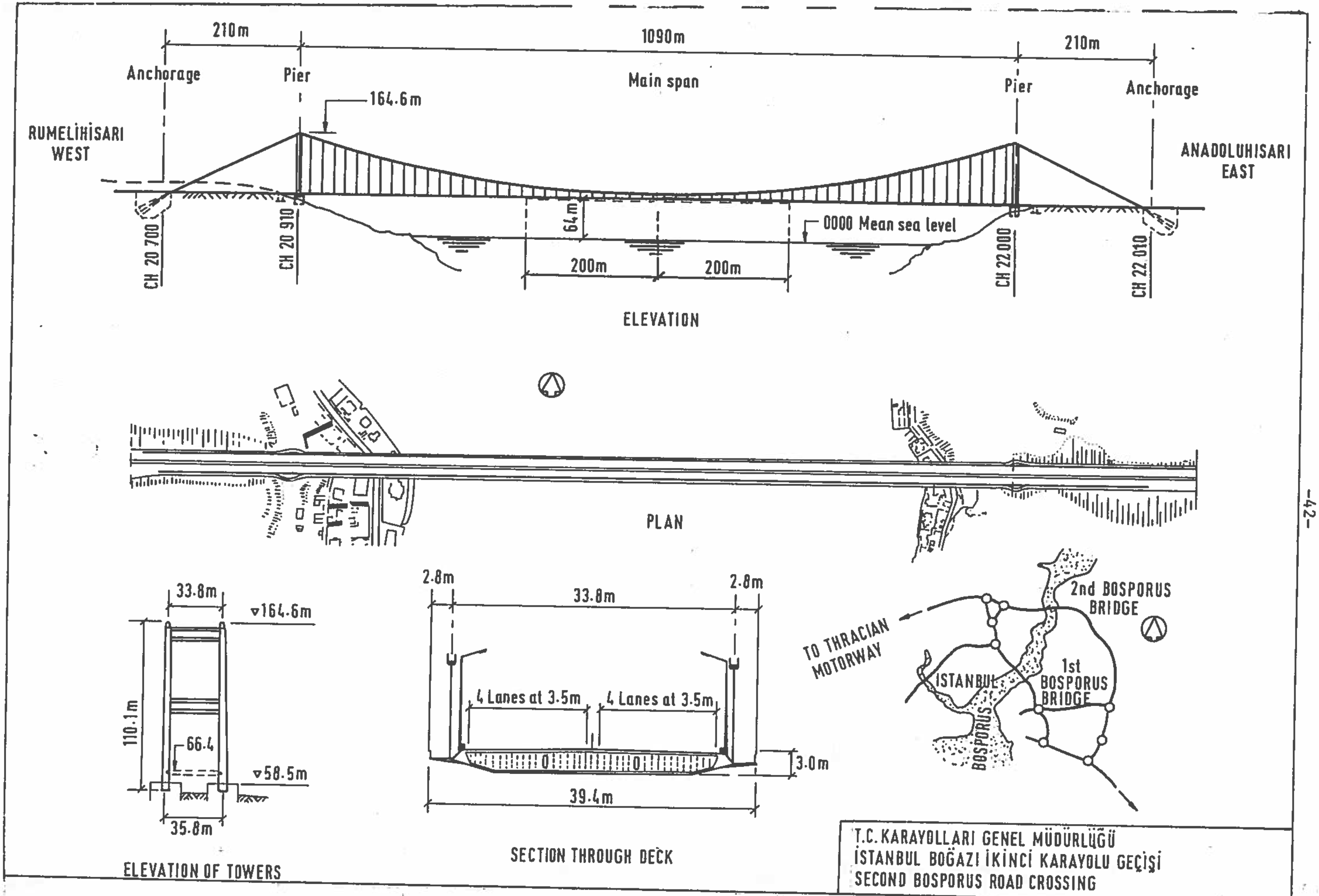

Fig. 1: General Arrangement of the Fatih Bridge.

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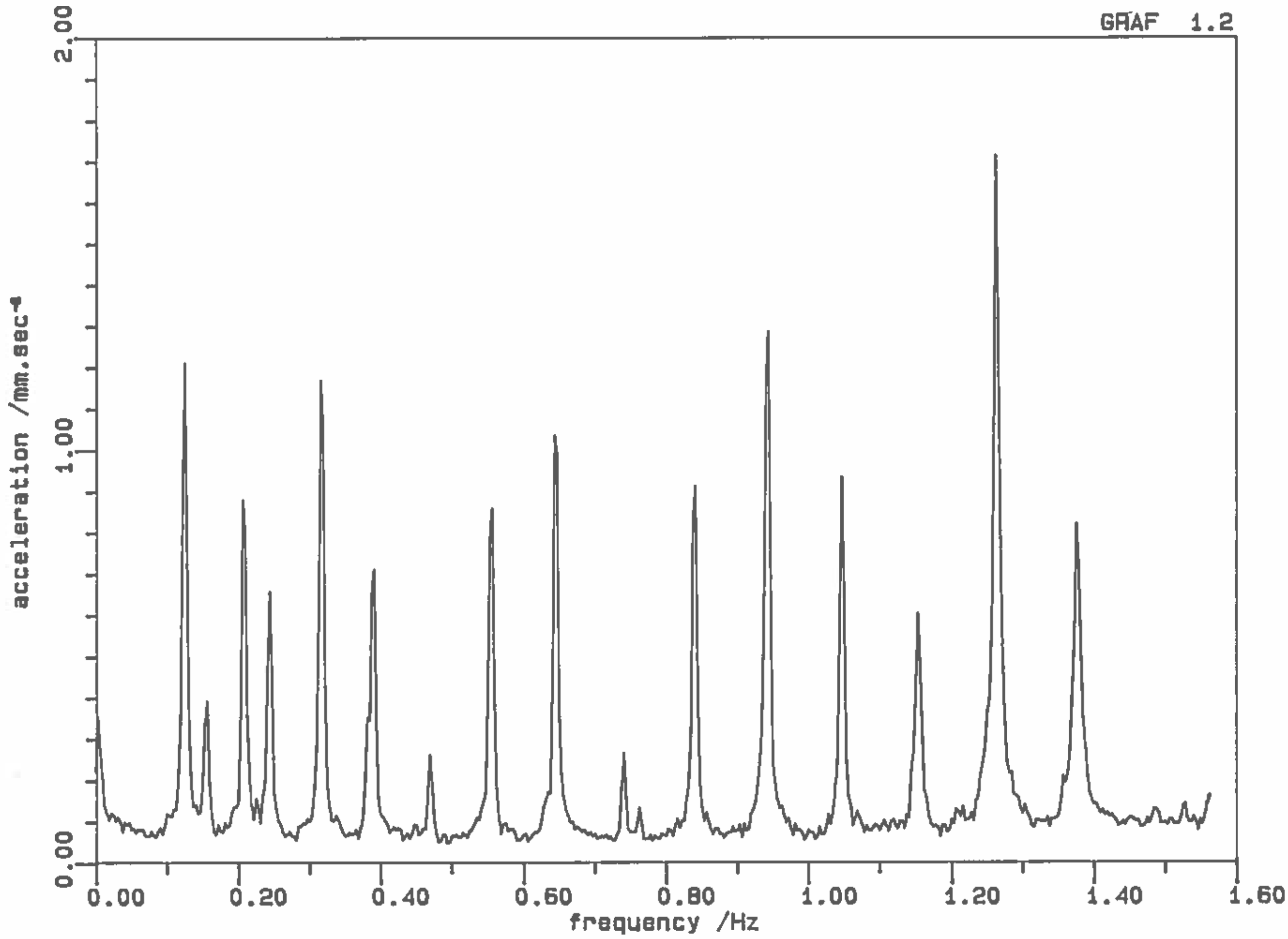
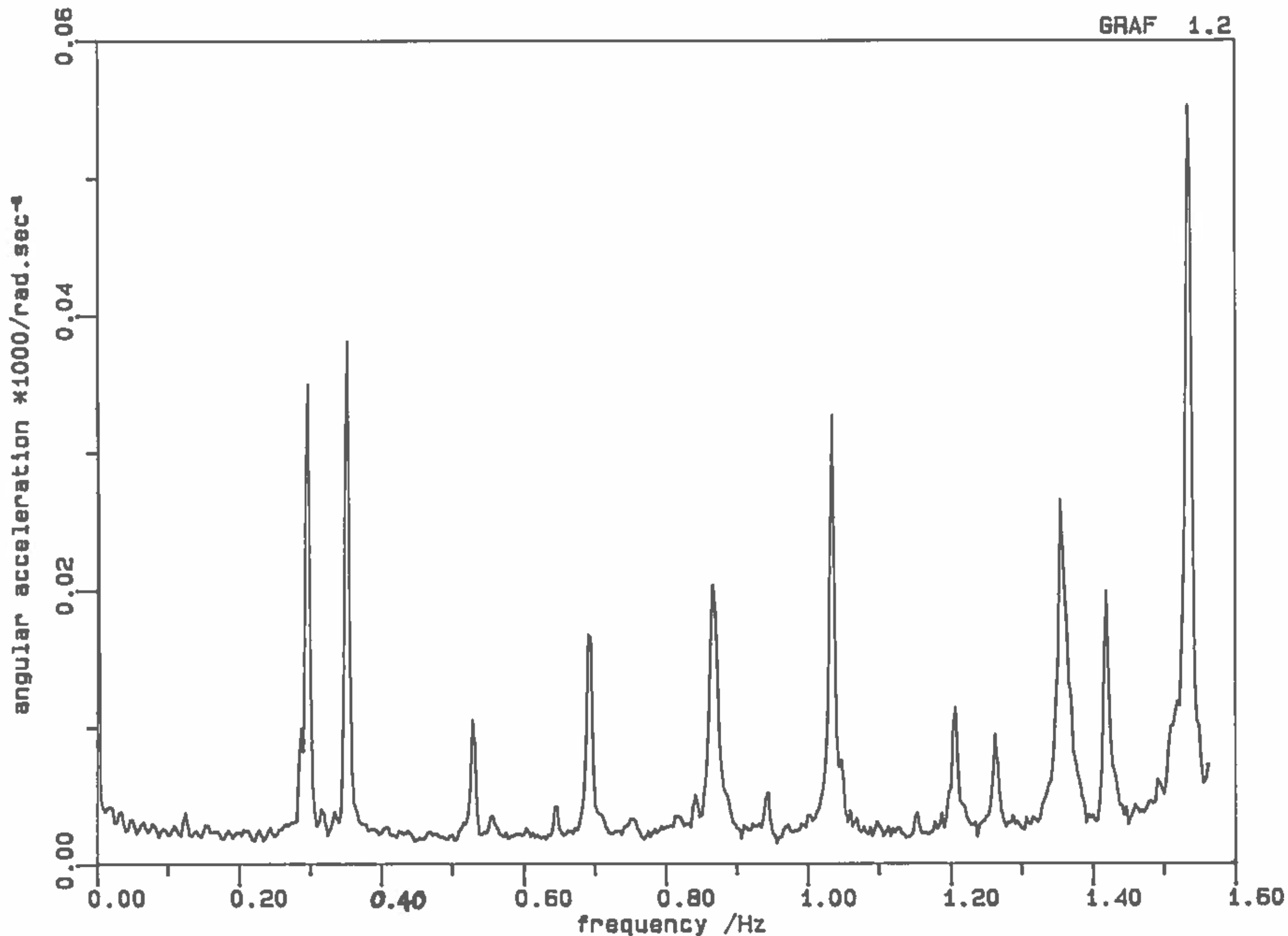


Vertical response at reference, (I1+I4)/2

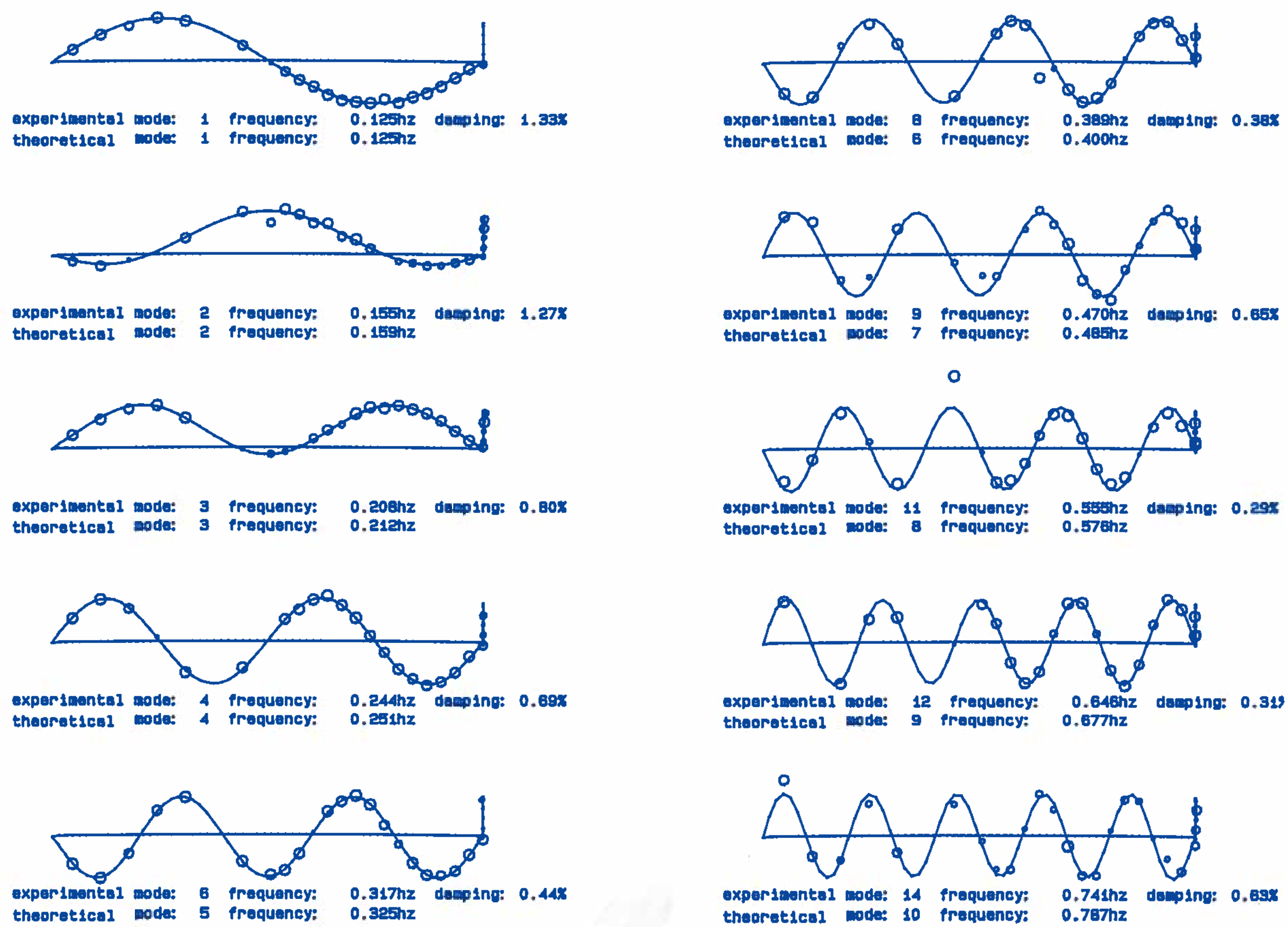


Torsional response at reference, (I1-I4)/2

Fig 2

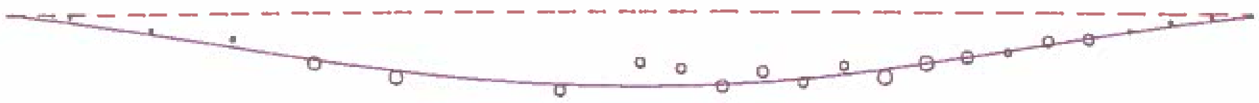
Fig 4

Fig 3

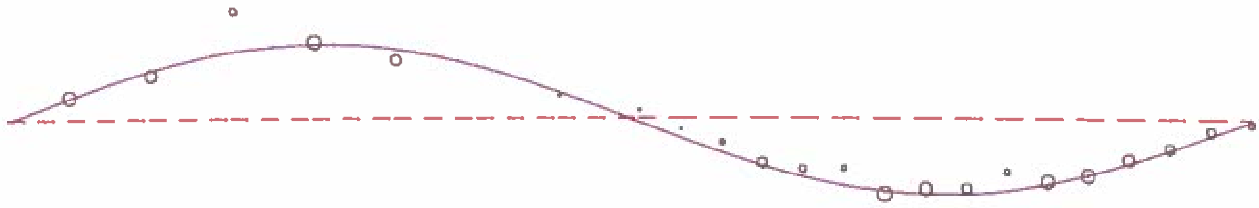


experimental mode: 1 frequency: 0.125hz damping: 1.33%
theoretical mode: 1 frequency: 0.125hz
experimental mode: 2 frequency: 0.155hz damping: 1.27%
theoretical mode: 2 frequency: 0.159hz
experimental mode: 3 frequency: 0.208hz damping: 0.80%
theoretical mode: 3 frequency: 0.212hz
experimental mode: 4 frequency: 0.244hz damping: 0.69%
theoretical mode: 4 frequency: 0.251hz
experimental mode: 6 frequency: 0.317hz damping: 0.44%
theoretical mode: 5 frequency: 0.325hz
experimental mode: 8 frequency: 0.389hz damping: 0.38%
theoretical mode: 6 frequency: 0.400hz
experimental mode: 9 frequency: 0.470hz damping: 0.65%
theoretical mode: 7 frequency: 0.485hz
experimental mode: 11 frequency: 0.555hz damping: 0.29%
theoretical mode: 8 frequency: 0.576hz
experimental mode: 12 frequency: 0.646hz damping: 0.31
theoretical mode: 9 frequency: 0.677hz
experimental mode: 14 frequency: 0.741hz damping: 0.63%
theoretical mode: 10 frequency: 0.787hz

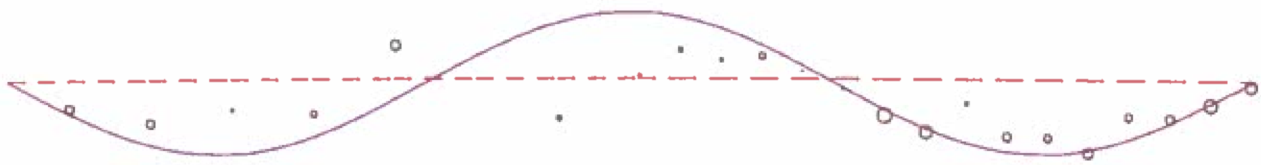
Fg 4



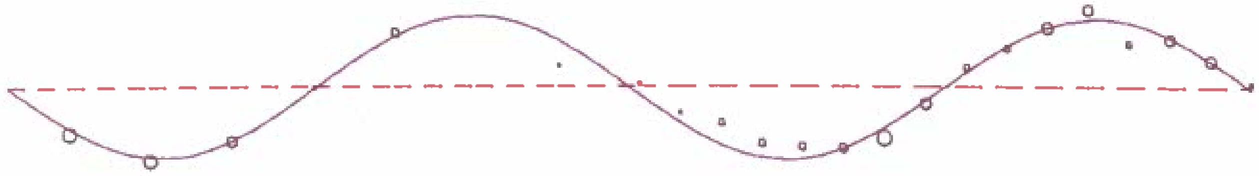
experimental mode: 5 frequency: 0.296hz damping: 0.70%
theoretical mode: 6 frequency: 0.243hz



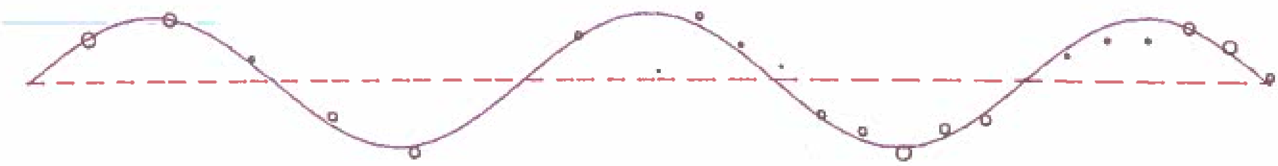
experimental mode: 7 frequency: 0.352hz damping: 0.77%
theoretical mode: 13 frequency: 0.333hz



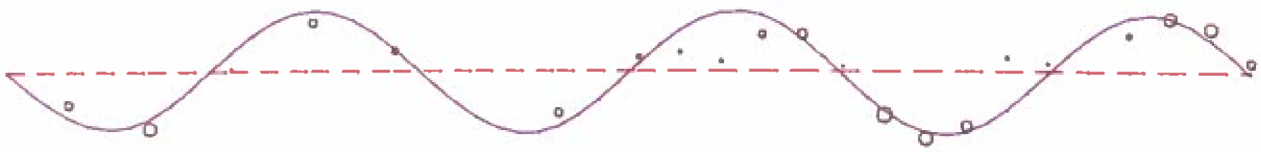
experimental mode: 10 frequency: 0.530hz damping: 0.56%
theoretical mode: 20 frequency: 0.501hz



experimental mode: 13 frequency: 0.693hz damping: 0.64%
theoretical mode: 26 frequency: 0.661hz



experimental mode: 16 frequency: 0.867hz damping: 0.54%
theoretical mode: 33 frequency: 0.828hz



experimental mode: 18 frequency: 1.036hz damping: 0.41%
theoretical mode: 40 frequency: 0.991hz

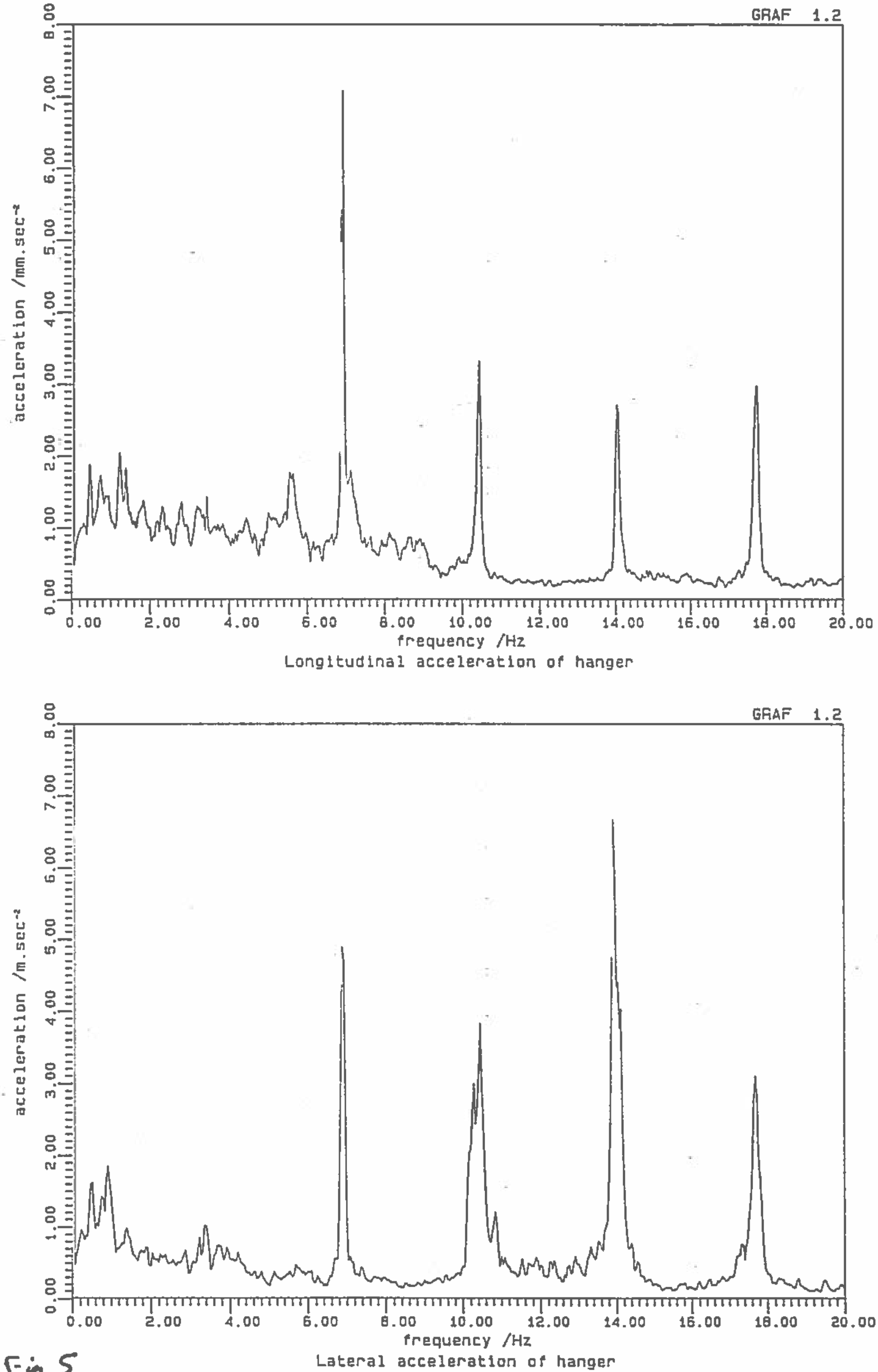
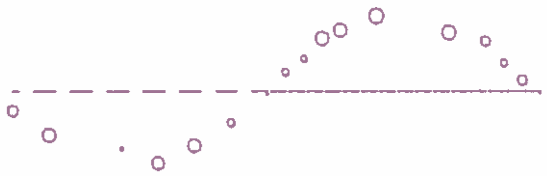


Fig 5 ~~Fig. 20~~ Auto power spectra of longitudinal and lateral response of hanger at reference (station 10)

Fig 6

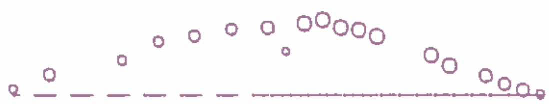
mode: 1 frequency: 0.077Hz damping: <14.47%

mode: 2 frequency: 0.239Hz damping: <3.04%

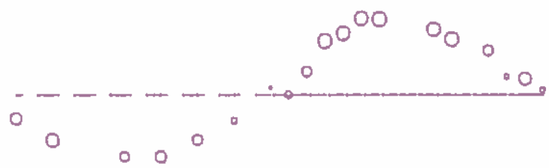


mode: 3 frequency: 0.250Hz damping: <2.37%

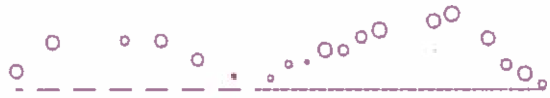
mode: 4 frequency: 0.287Hz damping: 0.97%



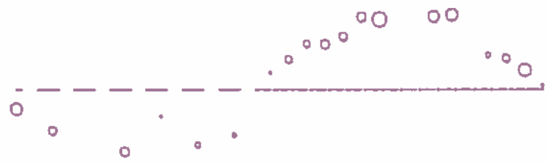
mode: 5 frequency: 0.296Hz damping: 0.73%



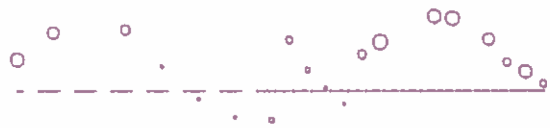
mode: 6 frequency: 0.315Hz damping: 3.51%



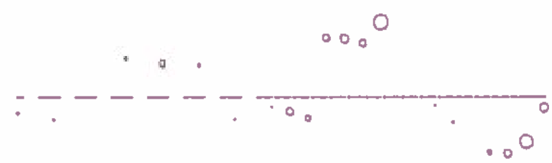
mode: 8 frequency: 0.432Hz damping: 1.39%



mode: 9 frequency: 0.466Hz damping: 0.90%



mode: 10 frequency: 0.504Hz damping: 0.78%



mode: 11 frequency: 0.868Hz damping: 0.70%