LOAD TESTING OF PONT SIR Y FFLINT

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INTRODUCTION

The LINK project 'Development of a Monitoring System for SMART Bridges' is scheduled to run from 1 February 1997 to 31 January 2000. The Consortium carrying out the work comprises TRL (Project Manager), Gifford and Partners, Dee Crossing Joint Venture, Flintshire County Council, Gage Technique, and Solartron. The programme of work is composed of seven tasks:

1. Preparation of Specification
2. Familiarisation with the Bridge Management System
3. Calculation of Predicted Response
4. Installation of Equipment
5. Monitoring, including load testing
6. Customise Bridge Management System
7. Reporting

This paper is concerned with Tasks 3, 4 and 5 and principally with load testing under Task 5. The load tests comprised static and dynamic testing.

Load testing can be carried out for a variety of reasons on existing and newly constructed bridges. For existing bridges, load testing can provide data about novel methods of design or construction and give assurance about the performance of the bridge. In the past, it was fairly common in the UK but has rarely been carried out in recent years. In contrast, new bridges are routinely load tested in some countries; for example Switzerland (1).

The Guidelines for Load Testing(2) define and distinguish the types of static load testing; supplementary, proof and proving. Supplementary load testing, as the name implies, is carried out to supplement numerical calculations and, most importantly, loads are sufficient to give measurable responses without causing permanent strain or damage. The tests described in this paper were supplementary to the designers' calculations.

Design of Pont Sir y Fflint, formerly and more commonly known as the Dee Crossing, has been described by Curran(3). The main structure in the Crossing is an asymmetric cable-stayed span of 194m length. Figure 1 shows a view of the completed structure. It is composed of in situ 8m segments having cable anchorages incorporated in the edge beams of each segment. Figure 2 shows a view of the underside of the bridge. The longitudinal edge beams and transverse beams are post-tensioned. The designer was mindful of the problems being experienced with some of the recently constructed concrete cable-stayed bridges in other countries in relation to the excessive vibration of the cables(4). Also the problems associated with grouting of post-tensioning ducts and corrosion of steel tendons (5).

One of the features of the bridge, decided at the design stage, was the incorporation of sensors to enable performance to be monitored during construction, and in the longer term. These comprised anemometers on the deck and tower, load cells fitted to selected cables, accelerometers on the deck and cables, and corrosion sensors at some of the more vulnerable locations. The LINK project included fitting of strain gauges during construction to the longitudinal beams, transverse beams and anchorage pockets. Outputs from the sensors are fed to data loggers, processed and fed to the TRL Bridge Management System. Periodic output reports can then be automatically provided to Flintshire County Council.

The objectives of the supplementary load testing are to:

- Obtain data for the structural response of the bridge immediately after construction, sometimes referred to as the ‘footprint’;
- Provide assurance of the design of the structure, for example levels of traffic-induced stress, dynamic response, etc;
- Check the performance of the sensors; and
- Generate fundamental data to aid the continued development of bridge design.

LOAD TESTING ARRANGEMENT

A total of 25 number 140mm long vibrating wire (VW) strain gauges were installed by Gage Technique (Task 4 of the Programme) for the static load test. The schematic position of the VW gauges is shown in Figure 3. In addition, four electric resistance strain (ERS) gauges and two tri-axial accelerometers were installed for the dynamic load test.

The static load test was carried out by placing eight 32 tonne pre-weighed aggregate lorries at pre-determined positions along the length of the bridge. The test was carried out according to a predefined and approved method statement, which consisted of 52 steps and was carried out with the help of 25 engineers, surveyors, technicians and drivers. Lorries were placed a pair at a time on the bridge and the following were monitored:

- Deck deflections
- Top of the tower horizontal movement
- Stay loads
- Stay anchorage pocket strains
- Longitudinal section strains
- Strains in the transverse beam

Figure 4 shows the positions where the deflections were monitored. The lorries were placed with their back axles back to back centred at monitoring positions on the main span. Figure 5 shows the arrangement of the eight lorries at one of the positions. The arrangement of lorries was applied on the line of stay positions where stay anchorage pockets were instrumented with VW gauges, at a section for longitudinal strains and on a transverse beam.

The dynamic load test was carried out by driving at 64km/h an instrumented lorry belonging to TRL which had been used in previous projects to measure its impact characteristics when travelling over differing road surfaces[6]. The lorry was driven over the eastbound carriageway eleven times and but only once over the westbound carriageway due to shortage of time. Planks of 25mm and 50mm thick were used to excite either wheel hop or body bounce impacts, respectively. The planks were placed close to sections where the ERS gauges were installed on the edge beams. The lorry was also driven over the bridge without a plank as a reference run.

The dynamic load tests were conducted on the afternoon of 4 March 1998 and followed by the static load tests, which finished at 4am on 5 March 1998. The bridge was opened a few days later by Her Majesty the Queen.

During the static load test the temperature of the deck was constant between 6 to 8 degrees Centigrade. However, high winds picked up over a short period of the testing work which when combined with rain resulted in significant disruptions.

STATIC LOAD TEST RESULTS

Significant amounts of data were collected during the static load test(7). In this section a comparison of selected measured values against the predicted is given. The predicted values were based on 2D and 3D computer models and concrete with an E value of 36N/mm^2. But subsequent tests on concrete samples indicated that the average value was 45N/mm^2.

The deck deflections and top of the tower horizontal movements under the full eight 32 lorries are compared with the predicted values in Tables 1 and 2. The ratios of the measured and predicted deck deflections were between 0.82 and 0.94 with an average value of 0.90. The ratios of the measured and predicted top of the tower horizontal movements were between 0.72 and 0.90 with an average value of 0.85.

The changes of stay loads under the full eight 32 lorries with the stay at the centre of the eight lorry arrangement are compared with the predicted values in Table 3. The ratios of the measured and predicted values were between 0.70 and 0.93 with an average value of 0.84.

Of the four anchorage pockets monitored, it was subsequently realised that results of three of them, which were on the north side of the deck, were affected by electromagnetic field from a generator which was parked close to the relevant duct. However, the stay anchorage pocket strains at M13 on the south side of the deck were not affected. Table 4 compares the measured strains and predicted values for the full load of eight lorries at this stay position. Gauges A and B were on the sides of the pocket and gauge C was at 1.5m in front of the pocket considered to be away from stress concentrations around the pocket (see Figure 6). The ratios of the measured to predicted values are between 0.89 and 1.17. The higher ratios are for the E value of 45kN/mm^2.

The strains in the longitudinal section of the deck under the eight lorries are summarised in Table 5. The predicted concrete strains for an E value of 45kN/mm^2 are also given in this table. VW gauges in top flange were in sensitive positions where small variations in location could cause large changes in strain. In consequence, output varied in relation to calculated values. In order to rationalise the data, ratios of measured to predicted values have been averaged; for the top slab, the average is 0.99 and for the bottom of the beam it is 0.95.

The strains in the transverse beam under four lorry loads are summarised in Table 6. The predicted values for the E value of 45kN/mm^2 are also included in this Table. It can be seen that the ratios of measured to predicted strains are rather variable. However, the higher strains measured close to the ends of the transverse beam indicate a higher degree of restraint by the main beams to the transverse beam. The lower strains measured at midspan are consistent with this observation. Summations of values of bottom strains are:

Measured = 38+(23+25)½ = 62 micro-strain

Predicted = 53+10.8 = 63.8 micro-strain

The close comparison of measured and predicted summations of strains confirms the equilibrium is satisfied and an E value of $45kN/mm^2$ is a reasonable representation of the response by the concrete to short duration loads. This overall behaviour indicates that the edge beams are torsionally stiffer than have been assumed in the computer models. A similar result should be obtained from the top strain gauges but, due to their closer proximity to the neutral axis of the section, their results are not as reliable as the bottom gauge readings.

DYNAMIC LOAD TEST RESULTS

The dynamic load test[8] indicated that the impact factors from the vehicle wheel measured by the vehicle instruments were between 1.7 and 2.0 with an average value of 1.84 for the 25mm profiled plank. The corresponding values for the 50mm thick plank were between 2.4 and 2.9 with an average value of 2.7. The impact factors without any plank were as low as 1.1 and 1.2.

The ERS gauges on the soffit of the edge beams, shown in Figure 7, indicated that the impact factors for the 25mm thick plank was between 1.1 and 2.8 depending on the lane on which the lorry was driving, and the distance of the gauges from the plank position. The corresponding values for the 50mm plank were between 1.4 and 2.9. However, the results of the 50mm thick plank may not be the true representation of the impact because the distance between the plank and the ERS gauges was too short to pick up the maximum effect. The impact factors without any planks were between 1.4 and 2.1. This is considered to be due to small changes in the stiffness caused by the transverse beams in the deck.

A fast Fourier transform was carried out on the deck data to produce a power spectrum. The results from the analysis of the wheel hop (25mm thick plank) tests showed a single defined peak at approximately 10-12Hz (Figure 8). The results from the body bounce (50mm think plank) tests exhibited several peaks with the lowest recorded frequency at 2.4Hz and other distinct peaks at approximately 5,9,11 and 12Hz (Figure 9). The designers' eigenvalue analysis of the structure had indicated that the first and second torsional modes occur at 0.895 and 1.26Hz. The lowest frequency of 2.4Hz measured during the test is about twice the second calculated torsional mode of vibration of the deck.

CONCLUSIONS

The static load test employed eight 32 tonne lorries located at predetermined positions on the deck of the 194m main cable-stayed span. Measured deflections and strains are compared; some unexpected differences are explained in terms of alternative structural actions.

- Vertical deck deflections were found to be, on average, 90 per cent of the predicted values.
- Horizontal tower deflections were, on average, 85 per cent of the predicted values.
- Forces in the main stay-cables of each load step were, on average, 84 per cent of the predicted values.
- Anchorage pocket strains were close to the predicted values.
- Longitudinal section strains were variable for the gauges in the top flange, due to VW gauge close proximity to the neutral axis of the section. However, their average was equal to predicted values.

- Transverse beam strains were different to the predicted values. However, their static totals were consistent with a simple redistribution of the moments from midspan to the ends of the beam. This was considered to be due to higher than expected restraint imposed by the connections between the longitudinal and transverse beams. Summations of the values of bottom strains provided a very close correlation. Top strains were more variable due to close proximity of the neutral axis.

The dynamic load test employed an instrumented 30 tonne lorry to excite the deck. The wheel impacts on the lorry and the deck were measured along with some of the excited natural modes of vibrations of the deck.

- The average wheel impact factors on the lorry were 1.15, 1.84 and 2.7 for the cases of no plank, 25mm and 50mm planks, respectively.
- The average impact factors on the edge beams were 1.77, 1.97 and 1.88 for the cases of no plank, 25mm and 50mm planks, respectively. However, the distance between the 50mm plank and the gauges were not adequate to pick up the maximum effect.
- The excitation energy from the impacts of the lorry on the planks was not adequate to mobilise the lowest torsional mode of vibration of the deck.

The supplementary load tests carried out on the bridge have obtained a "footprint" of the bridge, provided assurance of the design and confirmed that the majority of the sensors are functional. The bridge will be monitored in the future and comparative results will give assurances of continued serviceability.

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7 Mehrkar-Asl, S. "Static Load Test of the Pont Sir y Fflint". Gifford and Partners Report No.B0540A/R01, June 1998.

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Table 1. Deck vertical deflections for concrete E=36kN/mm^2

Step	Stay	Segment	Measured (mm)	Predicted (mm)	Ratio
4	M7	8	61.75	67.0	0.92
12	M10	11	81.25	90.5	0.90
20	M13	14	107.25	114.5	0.94
28	M16	17	91.00	111.5	0.82

Ratio = measured to predicted
Stay M7 is in the Main span, number 7 from the tower.
Steps 4, 12, 20 and 28 include all the eight 32 tonne lorries centred at the corresponding stay position.

Table 2 Top of the tower horizontal movements for concrete E=36kN/mm^2

Steps	Stay	Measured (mm)	Predicted (mm)	Ratio
4	M7	9.3	10.36	0.90
12	M10	15.3	17.20	0.89
20	M13	21.7	24.20	0.90
28	M16	18.0	24.96	0.72

Ratio = measured to predicted

Table 3. Typical stay forces for concrete E=36kN/mm^2

Step	Stay	Segment	Measured (kN)	Predicted (kN)	Ratio
4	M7	8	306.5	330.5	0.93
12	M10	11	296.0	353.5	0.84
20	M13	14	245.1	351.1	0.70
28	M16	17	269.1	300.5	0.90

Ratio = measured to predicted

Table 4 Strains in anchorage stay pocket M13

Gauge	Measured (10^{-6})	Predicted (10^{-6}) E=36kN/mm^2	Predicted (10^{-6}) E=45kN/mm^2	Ratio-1	Ratio-2
A	201	211	186	0.95	1.08
B	197	190	168	1.03	1.17
C	123	137	121	0.89	1.02

Ratio-1 = measured to predicted for E value of 36kN/mm^2
Ratio-2 = measured to predicted for E value of 45kN/mm^2

ıble 5 Strains at the longitudinal section

Position of VW gauge	Top beam North beam	Bottom beam North beam	Top quarter	Top centre lines	Top quarter	Top beam South beam	Bottom beam South Beam	ERS gauges on Soffit beam
Gauge No	VB16	VB17	VB18	VB19	VB20	VB21	VB22	-
Measured	-41.0	152.0	-34.0	-21.0	-56.0	-45.0	Lost	117.8
Predicted	-58.7	160.8	-29.7	-44.3	-29.7	-58.7	160.8	171.2
Ratio	0.70	0.95	1.14	0.47	1.89	0.77		0.69

Ratio = measured to predicted
Strains are in micro-strain (10^{-6})
Predicted for the E value of 45kN/mm^2

Table 6 Strains in transverse beam (under four lorries)

Position	Top south	Bottom south	Top midspan	Bottom midspan	Top north	Bottom north
Gauge	VB10	VB11	VB12	VB13	VB14	VB15
Measured	12	-23	-18	-38	12	-25
Predicted	3.0	-10.8	-20.0	53.0	3.0	-10.8
Ratio	4.0	2.13	0.90	0.72	4.0	2.31

Ratio = measured to predicted
Strains are in micro-strain (10^{-6})
Predicted for the E value of 45kN/mm^2

Figure 1 Pont Sir y Fflint

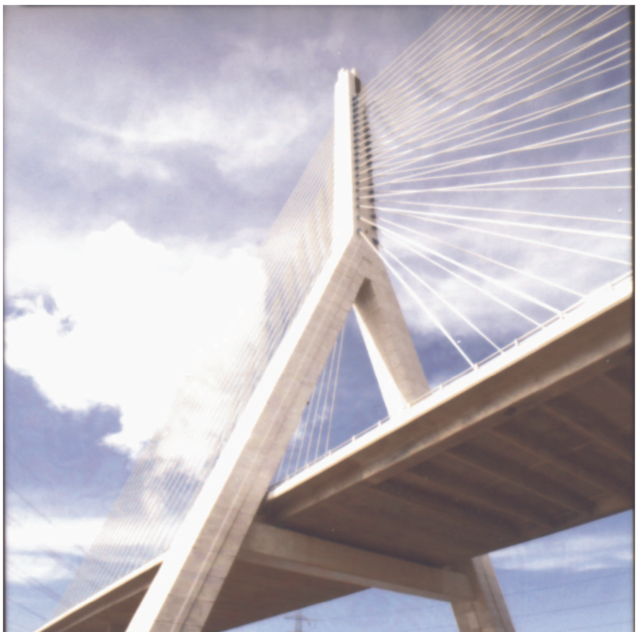

Figure 2 View from the banks.

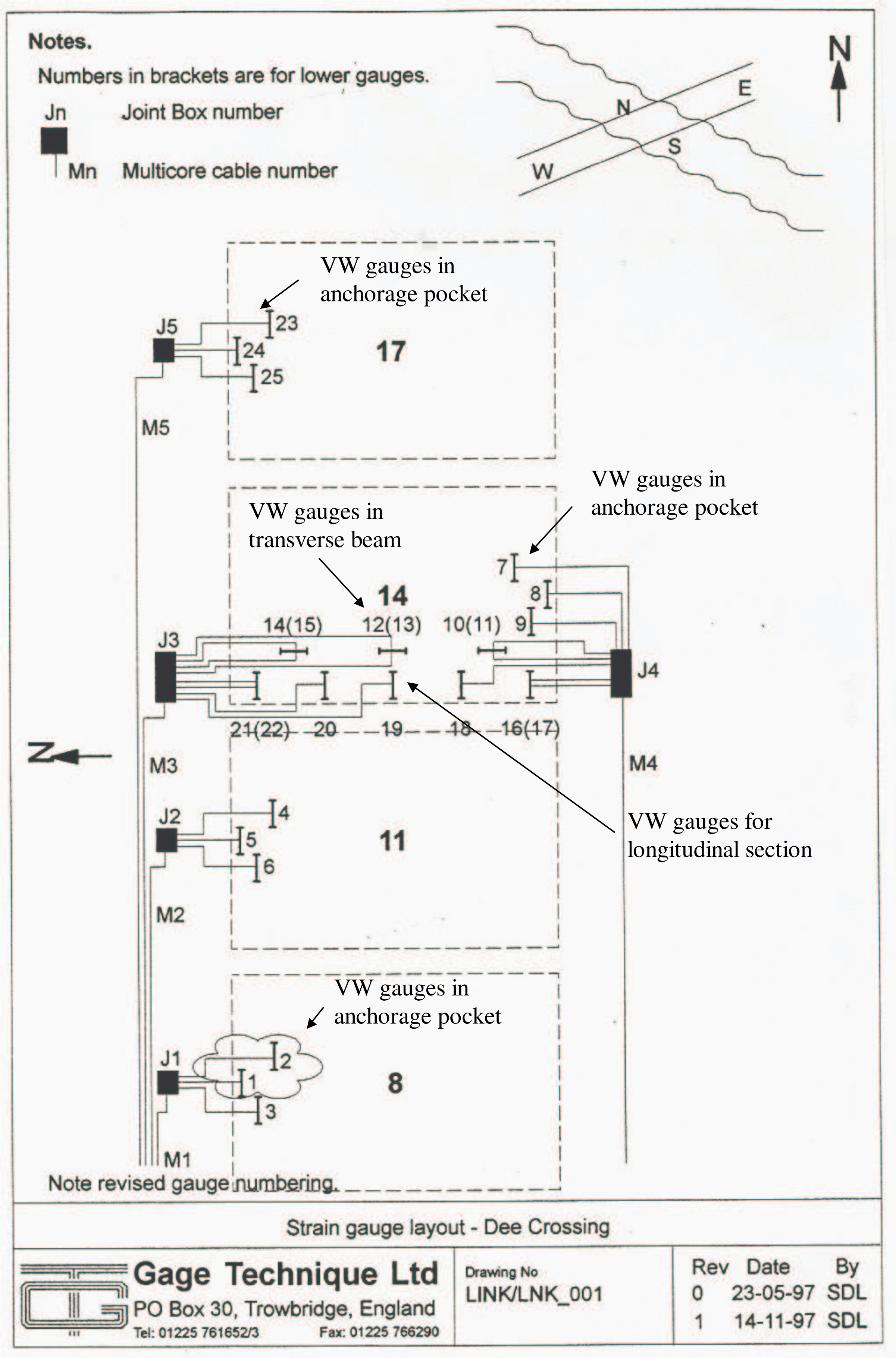

Figure 3 Schematic position of the VW gauges.

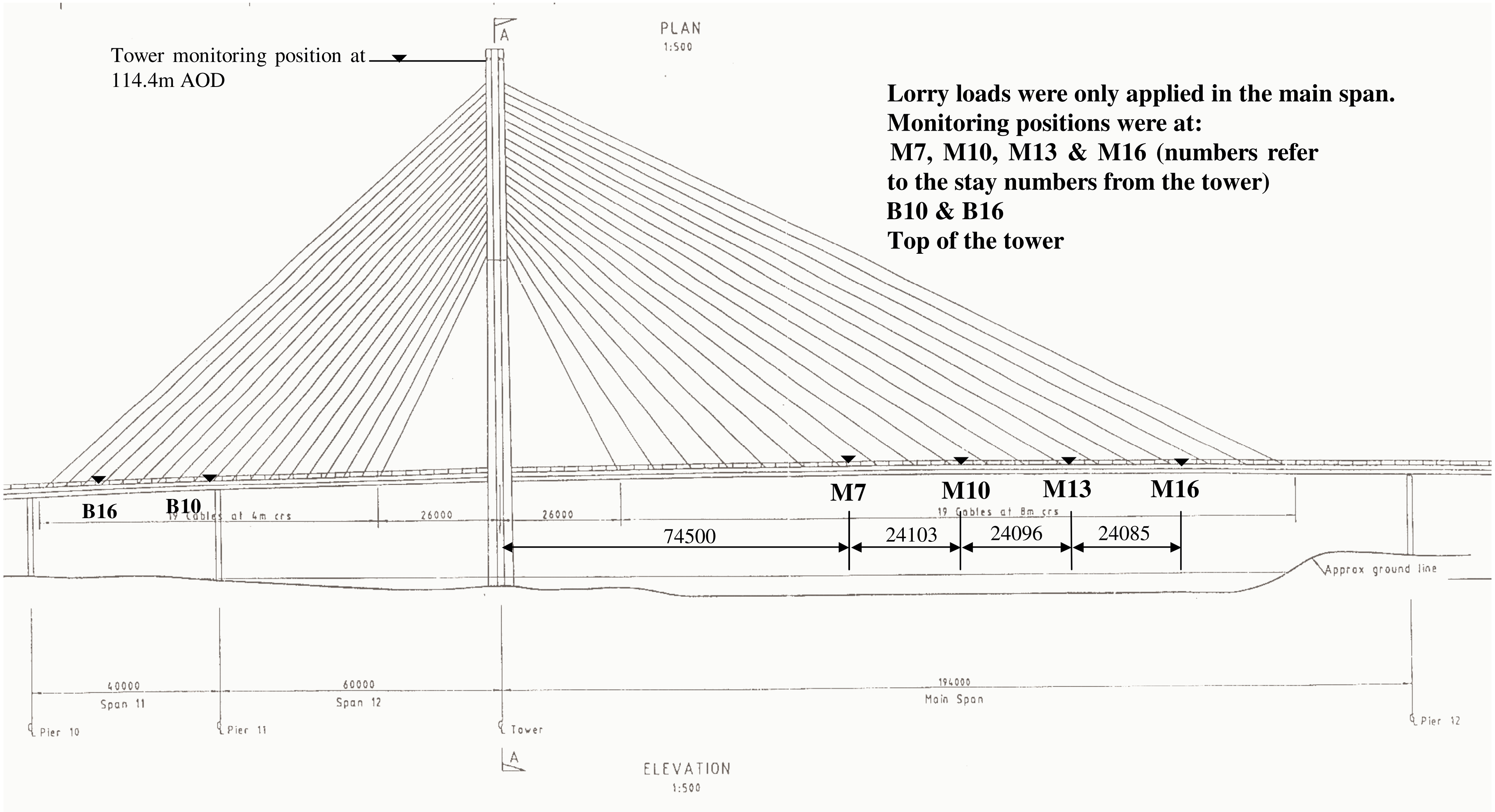

Figure 4 Monitoring positions along on the bridge

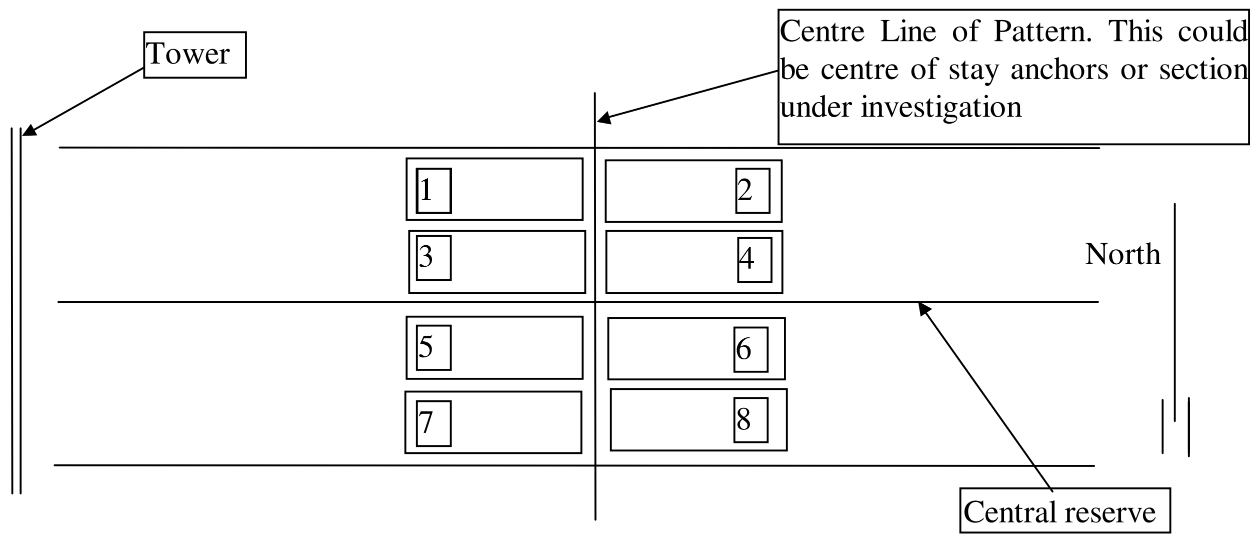

Figure 5 Loading pattern

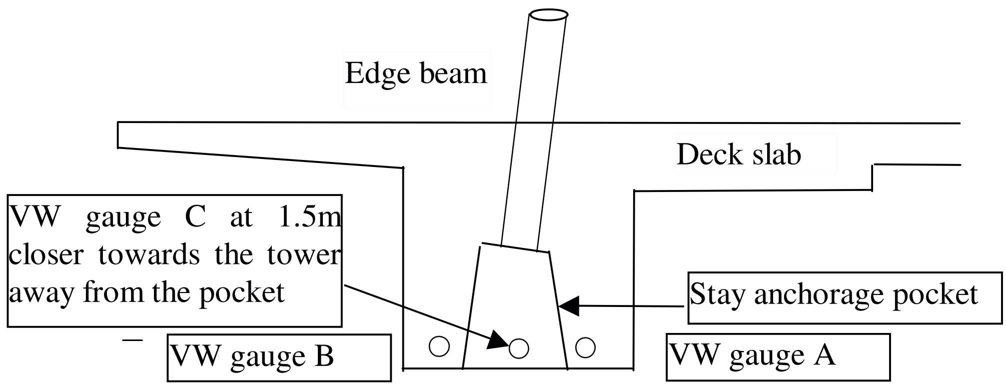

Figure - 6 VW gauges on sides of the stay pockets

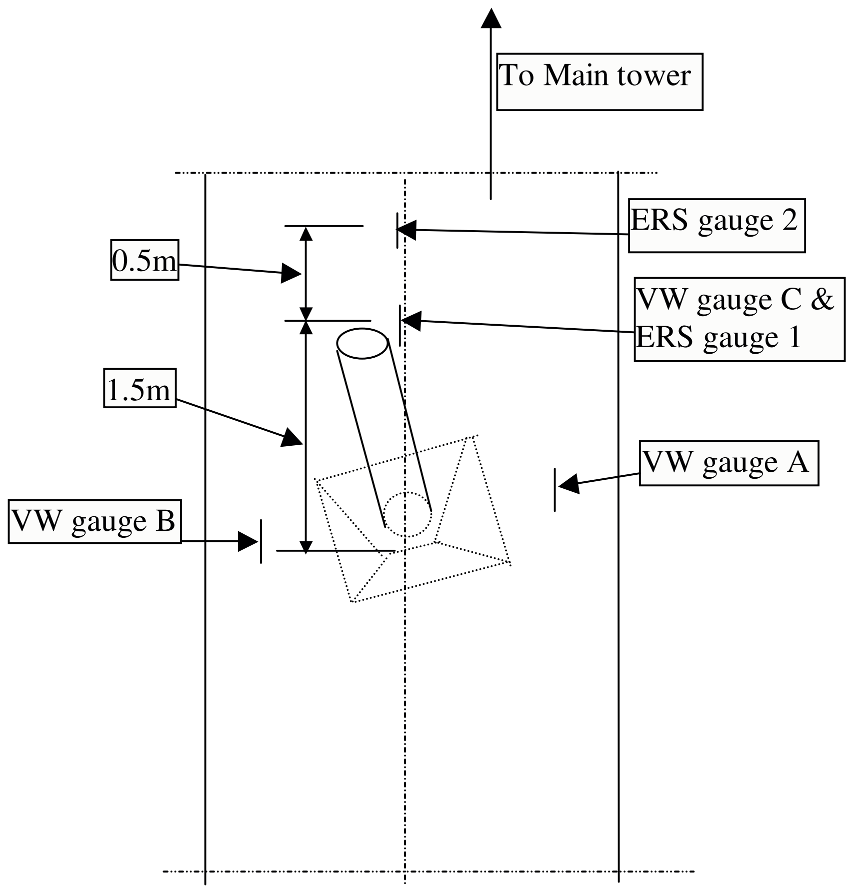

Figure 7 Plan view of the edge beam with VW and ERS gauges at anchorage stay pocket

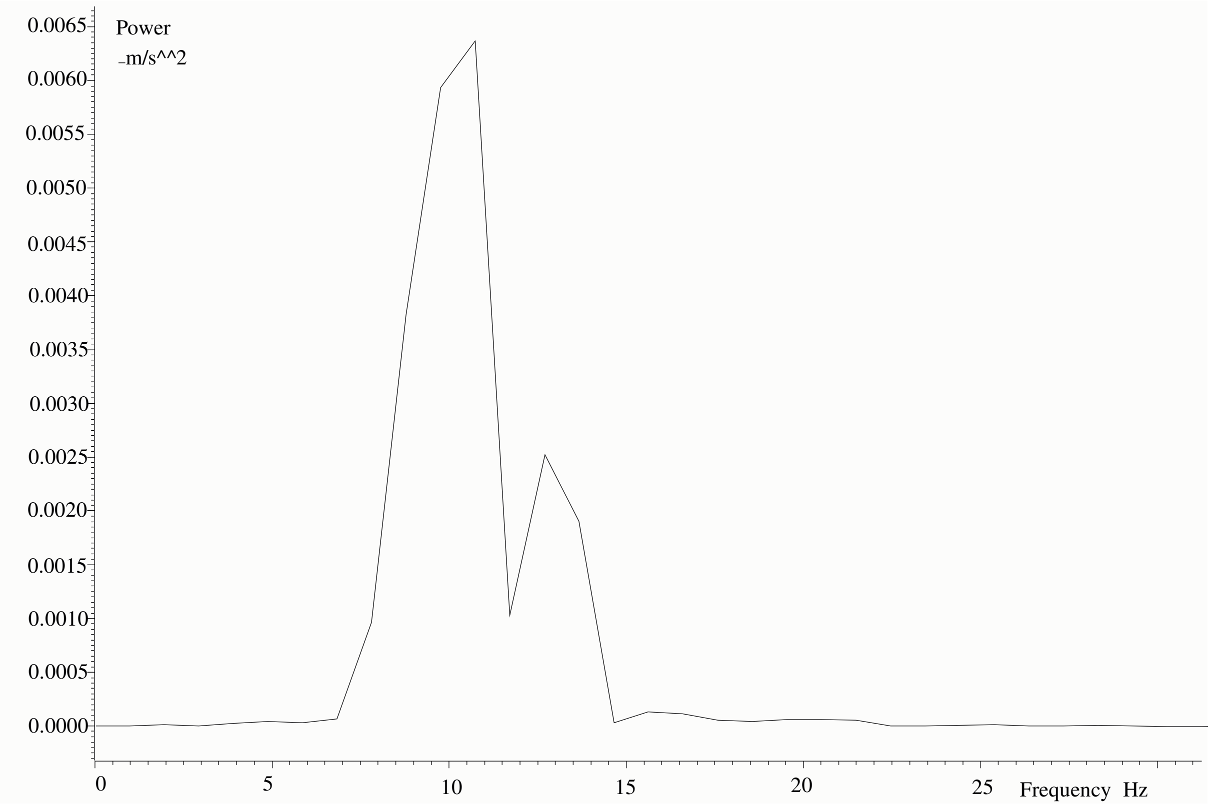

Figure 8 Deck accelerator data: example of power spectrum derived from the wheel hop test

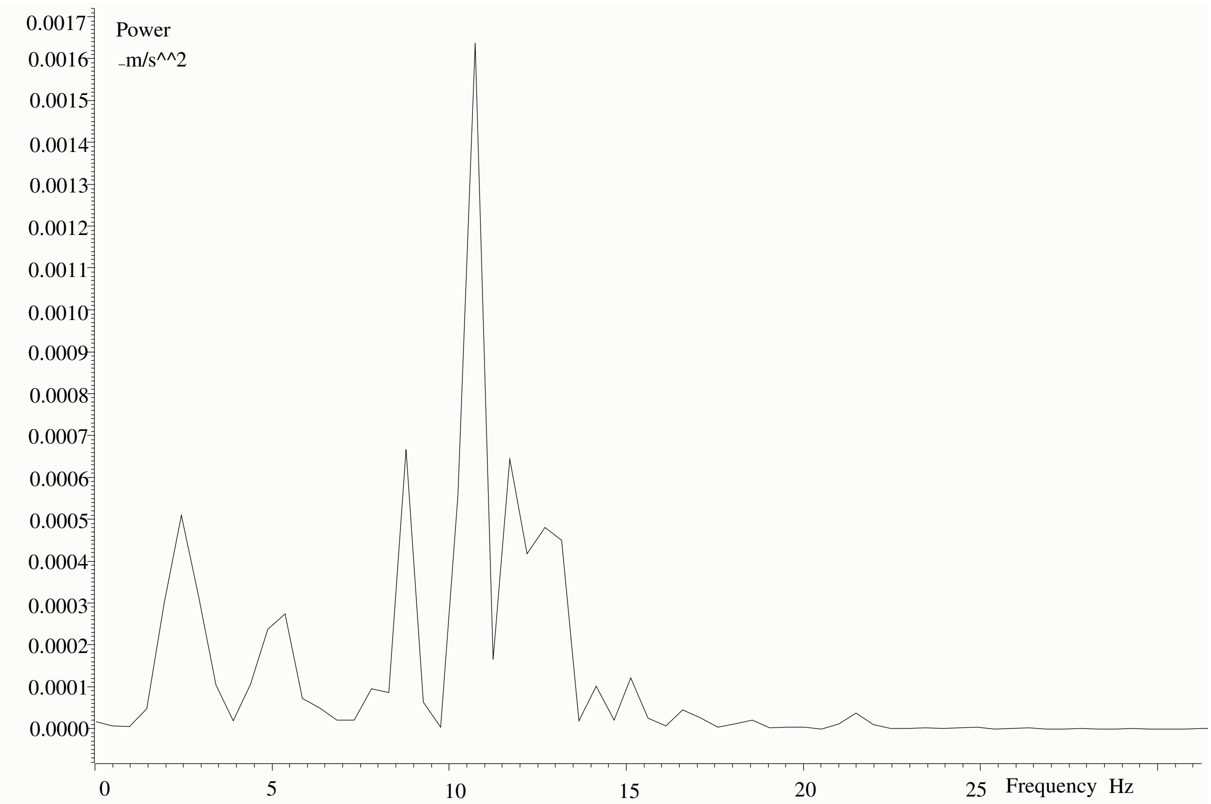

Figure 9 Deck accelerator data: example of power spectrum derived from the body bounce test