Health Monitoring of Superstructures of New Zealand Road Bridges:

Waipawa Bridge, Hawke's Bay

Transfund New Zealand Research Report No.170

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Waipawa Bridge, Hawke's Bay

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Executive Summary

Introduction

Bridge Health Monitoring is a method of evaluating the ability of a bridge to perform its required task (also called Fitness for Purpose) by monitoring the response of the bridge to the traffic loads it has to withstand.

This report is part of Stage 2 of a research project carried out in 1998-1999, which involves the *Short-Term Health Monitoring and "Fitness for Purpose" Assessment* of ten bridges on New Zealand highways, in order to develop and evaluate the methodology.

The Waipawa Bridge, on State Highway 2, crosses the Waipawa River near Waipawa, about 70 km south of Hastings, Hawke's Bay Region, North Island. It was selected as one of these ten, because it is relatively old (in service since 1958), it is a steel girder structure with spans that are longer than usual for this type of bridge, and its conventional bridge rating of 83% is less than 100%. The deck also has a low capacity factor (0.6), and the girder spacing is relatively wide.

The report details a theoretical assessment of the bridge to determine both the critical elements for the Health Monitoring programme, and the Fitness for Purpose Evaluation for the bridge based on the health monitoring data. This evaluation has only considered bending and shear of the main girders, and the bending capacity of the deck.

Theoretical Analysis

The theoretical analysis of the bridge found that the rating was 140% and the posting was 170%. This rating and posting were significantly higher than those in the 1999 Transit New Zealand Structural Inventory. This may be due to assumptions made regarding the degree of composite action and the inclusion of the kerbs in the strength calculations. The rating of the deck (0.74), based on plate bending, was similar to the rating from the Inventory, although the rating based on the empirical method was higher. Based on these results, the Health Monitoring programme concentrated on the performance of the girders and the deck slab.

Health Monitoring Results

The Health Monitoring investigation found that:

- Although the spans were assumed to behave as simply supported spans, some continuity occurred between the span that was monitored and the adjacent span.
- The heaviest vehicles are typically inducing effects around 95% to 100% of the 0.85 HN* loading, and a significant portion of the vehicles on this route are lighter than these vehicles. Overloading is well controlled on this route.
- Some differences between the analytical distribution of load and the recorded distributions of load could be related to differences in vehicle position in the lane adjacent to the footpath.

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* HO Highway overweight vehicle; HN Highway normal vehicle

- The impact factor of 1.25 obtained from the Bridge Manual is appropriate for this bridge. The significant differences in the values obtained for vehicles travelling in different directions may be related to the effect of the road profile.

Fitness for Purpose Evaluation

- The Fitness for Purpose Evaluation for this bridge, based on the critical midspan bending, was 180%. This indicates that the bridge is safely carrying the heavy vehicle traffic using the route with significant strength reserves.
- The transverse cracking in the deck slab is not significantly affecting the performance of the slab and the Fitness for Purpose Evaluation of the deck slab was 135%, or 1.35 in terms of Deck Capacity Factor.

Recommendation

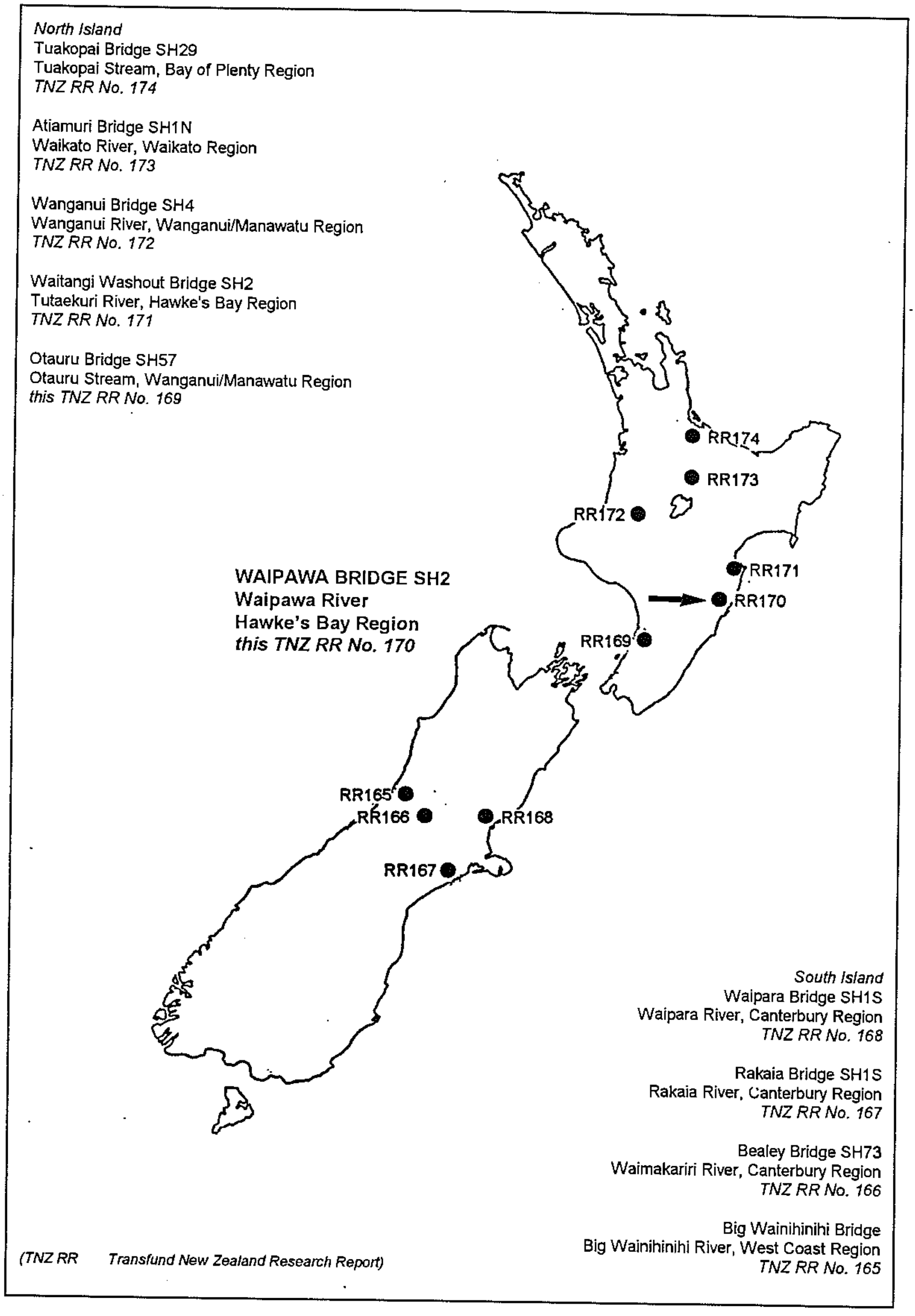
The cracking of the deck should be addressed only from a durability perspective to minimise future bridge maintenance costs and to prolong the life of the bridge.

Abstract

Bridge Health Monitoring is a method of evaluating the ability of a bridge to perform its required task (also called Fitness for Purpose) by monitoring the response of the bridge to the traffic loads it has to withstand.

This research project, carried out in 1998-1999, is part of Stage 2 of the *Short-Term Health Monitoring and "Fitness for Purpose" Assessment* of ten bridges on New Zealand highways, in order to develop and evaluate the methodology. The Waipawa Bridge, on State Highway 2, crosses the Waipawa River about 70 km south of Hastings, Hawke's Bay Region, North Island. It was selected as one of these ten, because it is relatively old (in service since 1958), is a steel girder structure whose spans are longer than usual for this type of bridge, and the girder spacing is relatively wide. The Fitness for Purpose Evaluation derived from ambient heavy vehicle traffic suggests that the bridge is performing better than the theoretical evaluation suggests. Reasons for this improved performance are discussed in the report.

Figure 1.1 Location of Waipawa Bridge, North Island, New Zealand, one of the ten bridges selected for the Bridge Health Monitoring project.



1. Introduction

1.1 Background

Bridge Health Monitoring is a method of evaluating the ability of a bridge to perform its required task, also called its "Fitness for Purpose". This method involves monitoring the response of a bridge to its normal environment, in particular to the traffic loads it has to withstand. Subsequently this data is processed and used to evaluate the bridge's Fitness for Purpose.

Bridge Health Monitoring requires a hybrid mix of specifically designed instrumentation technology and data processing, with conventional bridge theory and evaluation techniques. It has not been previously used in New Zealand as a systematic bridge evaluation technique, and consequently a project was conceived with the following objectives:

- To develop an appreciation of a sample of the existing New Zealand bridge infrastructure;
- To develop rational guidelines for evaluating the Fitness for Purpose of New Zealand road bridges based on sound engineering principles;
- To identify and understand the reasons for differences between the Fitness for Purpose Evaluation and traditional analytical ratings;
- To provide validation and data inputs for improving bridge design and evaluation procedures.

This project, conducted in 1998-1999, was divided into four stages, of which Stage 2 was entitled *Short-term Health Monitoring and "Fitness for Purpose" Assessment.* Short-term Health Monitoring was conducted on a total of ten New Zealand bridges on state highways, covering a range of bridge types, ages, conditions and environments. This population of ten bridges was selected to be representative of the New Zealand bridge population. It thus provided an appropriate basis to compare conventional bridge evaluation with the bridge Health Monitoring techniques under development. Not every aspect of every bridge has been considered, but rather the monitoring has typically focused on critical components of the superstructure of each bridge.

This report is part of Stage 2 of the project, and presents results for the Waipawa Bridge, on State Highway (SH)2. It crosses the Waipawa River near Waipawa, about 70 km south of Hastings, Hawke's Bay Region, North Island of New Zealand (Figure 1.1). The reasons for choosing this bridge for the representative sample were:

- It is relatively old (built in 1958).
- It is a steel girder structure whose spans are longer than usual for this type of bridge.

- Its Bridge Classification is 83%, (i.e. less than 100%).
- Its deck has a very low capacity factor (0.6).
- Its girder spacing is relatively wide and the girders are stiffened web plate girders.

The objective of this investigation was to evaluate the Fitness for Purpose of the superstructure of the Waipawa Bridge using the conventional evaluation technique and the proposed Health Monitoring technique, and to compare the results of both techniques. The fitness of the bridge to carry heavy vehicle traffic loadings was specifically investigated.

1.2 Applying Health Monitoring Technology

The Transit New Zealand Bridge Manual (TNZ 1994) procedure was used to complete the conventional evaluation. The Health Monitoring procedure involved the following steps:

- Performing a structural analysis on the superstructure of the bridge to determine the critical mode of failure and to determine the locations for health monitoring instrumentation.
- Monitoring the response of the structure to the ambient heavy vehicle traffic passing over the bridge for at least 24 hours (Health Monitoring).
- Recording the response of the structure to the passage of a heavy vehicle of known mass and dimensions to provide a reference for the health monitoring data.
- Evaluating the Fitness for Purpose of the superstructure based on health monitoring data, and comparing this with conventional evaluation methods.

The Fitness for Purpose Evaluation of the Waipawa Bridge is based principally on the following components of the superstructure:

- Midspan bending capacity of the main steel girders.
- Shear capacity of the main steel girders.
- Transverse bending capacity of the concrete deck.

The substructure was not evaluated as part of this investigation.

2. Evaluation of Bridges using Health Monitoring Techniques

2.1 Introduction

This section looks at the traditional approach to evaluating bridges as set out in the Bridge Manual (TNZ 1994). The advantages of a Health Monitoring approach are outlined, and a method to integrate the advantages of Health Monitoring in the existing evaluation procedures is also proposed.

Both bridge design and bridge evaluation involve ensuring that the probability of the load being greater than the resistance (i.e. the bridge fails) is acceptably small. This is illustrated graphically on Figure 2.1.

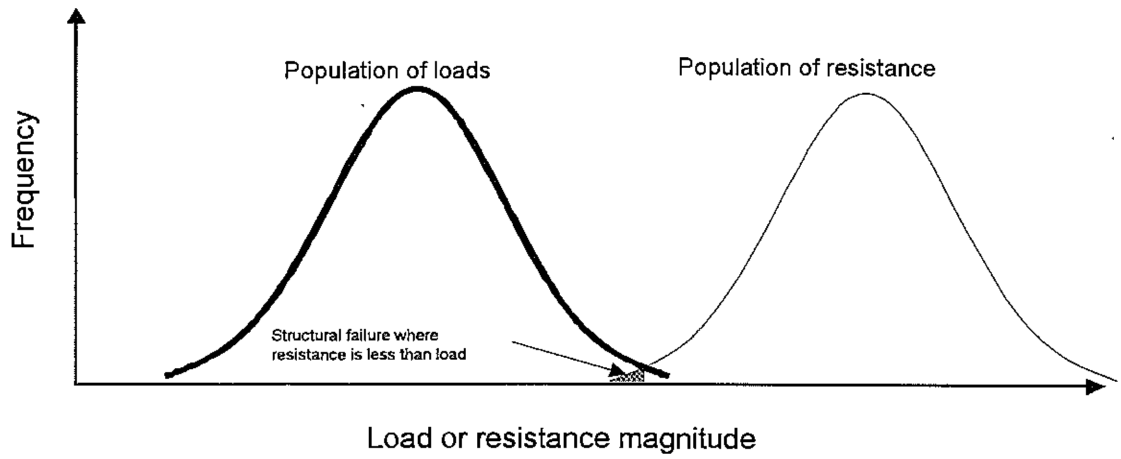


Figure 2.1 Statistical representation of structural failure.

Normally theoretical models are used to predict the magnitudes of loads and resistances in both design and evaluation processes. However, Health Monitoring utilises ambient traffic to investigate the effect that actual loads have on the in-situ structure. Thus the results of Health Monitoring provide an integrated measure of both the actual loads applied to the structure, and the effects that these loads have on the structure.

The objectives of bridge design and evaluation are similar, however the processes differ in some significant ways including:

- Bridge evaluation is more constrained than bridge design, since the infrastructure already exists in the latter case;
- Constraints are better understood during evaluation compared to design;
- Evaluation is usually associated with shorter time spans (typically 20 years compared to 100 years);
- Management options are often available and well understood during evaluations.

The estimation of structural resistance usually applies theoretical models based on engineering mechanics. Models of various levels of complexity are available, and these produce estimates of capacity with different levels of accuracy. Input data (material strengths, boundary conditions, etc.) are required for theoretical models, regardless of the model chosen. Much of these input data are based on a knowledge of construction procedures and tolerances. In the case of design, specific tolerances and parameters can be specifically controlled and confirmed where necessary.

When conducting evaluations however, greater uncertainty is usually associated with parameters (for example material strength). Conservative values can be chosen for the input data to allow for this, but will lead to under-estimation of capacity. Uncertainty may be reduced by testing all or part of the structure in some cases. Testing may also be important, because the resistance of an existing structure may decrease with time as physical deterioration progresses. In significantly deteriorated structures, this must be accounted for in the evaluation process.

Quantification of representative loads is generally more difficult than quantification of resistance, mainly because there is less control over bridge loading than there is over bridge construction and maintenance. In addition, design loads and legal loads are at best only indirectly linked. Design loads are generally developed by code writers who consider the worst-case loads likely to occur within the design life of structures. These loads are normally considered in two categories. The first is a set of loads intended to represent worst-case effects from normal legally loaded heavy vehicles (HN loading; TNZ 1994). The second is a set of loads intended to represent the worst-case effects from overloaded but permitted vehicles (HO loading; TNZ 1994). New bridges and their components are designed for the most severe effects resulting from both HN and HO loadings. This approach is intended to ensure that new bridges can accommodate current and foreseeable legal loads.

When evaluating existing bridges, there is limited scope to modify a bridge to change its capacity to accommodate future loads. However there is a strong need to understand its capacity to accommodate existing legal loads. The New Zealand Bridge Code (in TNZ 1994 Bridge Manual) empirically links legal loads with design loads for evaluation purposes. Essentially bridge evaluation loads are 85% of the design loads. If a bridge evaluation reveals that a given bridge cannot safely sustain 85% of the HO (overloaded/permitted legal heavy vehicle) loading, it will be **rated** consistent with its actual capacity to resist load. This rating will not be publicised, but will be used to approve or reject permit applications from transport operators requesting permission to cross the bridge with an overloaded (permitted) heavy vehicle. If a bridge evaluation reveals that a given bridge cannot safely sustain 85% of the HN (normal legal heavy vehicle) loading, it will be **posted** with a load limit consistent with its actual capacity to resist load.

2.2 Bridge Manual Evaluation Procedure

The Bridge Manual (1994) sets out the criteria for the design of new structures and evaluation of existing structures. Evaluation of existing structures is dealt with in Section 6 of that Manual. Existing bridges are typically evaluated at two load levels which are outlined below:

1. *A **Rating Evaluation** base on parameters to define the bridge capacity using overload factors and/or stress levels (i.e. appropriate for overweight vehicles).*

This evaluation is primarily concerned with evaluating the bridge's ability to carry overweight permit vehicles that comply with the Transit New Zealand Overweight Permit Manual (TNZ 1995), in a consistent and logical manner. However it is also used as a means of ranking and evaluating bridges for their capacity. This evaluation involves assessing the bridge's ability to carry a specific overweight vehicle load (0.85 HO loading).

2. *A **Posting Evaluation** based on parameters to define the bridge capacity using live load factors and or stress levels (i.e. appropriate for conforming vehicles).*

This evaluation is primarily concerned with evaluating the bridge's ability to carry vehicles which are characteristic of typical heavy vehicle traffic and comply with the TNZ Overweight Permit Manual (TNZ 1995). The evaluation involves assessing the bridge's ability to carry a design loading which is somewhat characteristic of typical heavy vehicle traffic (0.85 HN loading). If the bridge is unable to carry this loading, then the bridge is posted with the allowable load that the bridge can safely carry.

2.3 Member Capacity & Evaluation using TNZ Bridge Manual Criteria

The Bridge Manual deals with main members and decks of a bridge separately. The evaluation approach described in Section 6 of the Manual is summarised here.

2.3.1 Main Members

Equation 1 calculates the available vehicle live load capacity (or overload capacity) for a particular component of the bridge. This is the capacity available to carry unfactored service loads. A value of 1.49 for the overload factor is used for rating evaluations and a value of 1.9 is used for posting evaluations (TNZ 1994). These factors reflect the degree of uncertainty associated with the actual vehicle loads that will be applied to the bridge in each case. The higher the number the greater the degree of uncertainty.

$$R_o = \frac{\phi R_i - \gamma_D(DL) - \sum(\gamma(Other\ \ Effects))}{\gamma_o} \qquad \text{(Equation 1)}$$

where:

R_o = Overload Capacity		DL = Dead Load Effect
ϕ = Strength Reduction Factor		γ = Load factors on other effects
R_I = Section Strength		γ_o = Overload Factor
γ_D = Dead Load Factor		

2.3.1.1 *Rating Evaluations*

From the overload capacity, the ability of the bridge to carry the desired loads (Class) is calculated from Equation 2 which divides the Overload Capacity by the Rating Load Effect. The rating load effect is the effect of the evaluation vehicle on the bridge (85% of the HO) including the effects of eccentricity of load and impact. A value of 100% for the Class represents a bridge which can safely withstand the applied loads according to the Bridge Manual. Values of Class greater than 120% are recorded as 120%. The final Load Rating is found by first determining the Class for each girder (main component). The minimum Class then becomes the rating for that bridge.

$$Class = \left(\frac{R_o \times 100}{Rating\ Load\ Effect} \right)\% \qquad \text{(Equation 2)}$$

2.3.1.2 *Posting Evaluations*

A similar formula (Equation 3) applies for posting evaluations, with the Posting Load Effect represented by 85% of the 0.85 HN vehicle loading, including the effects of eccentricity of load and impact. There is an allowance for reducing impact if speed restrictions apply or are imposed.

$$Gross = \left(\frac{R_L \times 100}{Posting\ Load\ Effect} \right)\% \qquad \text{(Equation 3)}$$

2.3.2 Decks

The general principles for assessing the capacity of the deck to resist wheel loads are similar to those for the main members.

The Bridge Manual sets out procedures for calculating the strengths of concrete and timber decks, and the various wheel loads to be considered.

Generally the deck is then assessed based on similar principles to the main members along the lines of Equation 4, with the output being a DCF (Deck Capacity Factor). A DCF of 1.0 represents a deck which can safely resist the applied loads using the criteria in the Bridge Manual.

$$DCF = \left(\frac{Overload\ Capacity\ of\ Deck}{Rating\ Load\ Effect} \right) \qquad \text{(Equation 4)}$$

2.4 The Health Monitoring Approach

2.4.1 Theory of this Approach

As outlined in section 1 of this report, Health Monitoring is a method of evaluating the ability of a bridge to perform its required task, or Fitness for Purpose, by evaluating the response of the bridge to its loading environment.

Traditional methods of evaluation, as outlined in section 2.3, use a design load to represent vehicle effects (which may or may not accurately represent the traffic) and a series of factors to represent other load-related factors. There is also a series of assumptions regarding the strength of the structure and how it resists the loads.

Health Monitoring, which involves monitoring the response of the bridge to the ambient heavy vehicle traffic, has the advantage of measuring and considering the overall system including the bridge, road profile, type of traffic and the level of overloading. In fact, Health Monitoring of the bridge allows the influence of all these factors to be assessed for a specific site. By monitoring the response of the bridge for a short period of time and extrapolating these results using statistical and probability techniques, the health or Fitness for Purpose of a bridge can be assessed.

The Bridge Manual is based on limit-state design principles with the requirement for bridges to be designed for both strength and serviceability. For the purpose of assessing the probabilistic effects of loading, the Bridge Manual recommends a design life of 100 years. If the traffic effects were recorded for 100 years on a bridge, then the full spectrum of loads applied to the bridge would be measured and the bridge's ability to withstand these loads could be assessed.

Obviously, measuring the traffic effects for 100 years is not feasible or practical. Monitoring the traffic effects for a short period of time and extrapolating these data using statistical and probability methods provides an economic and viable alternative for assessing a bridge. Stage 3 of this research project will quantify the appropriate duration for monitoring, but this Stage 2 is based on short-term monitoring, and previous experience has shown that 1 to 3 days is normally an adequate period for Health Monitoring purposes.

Extrapolating short-term health monitoring data for periods of time that are representative of the design life of the bridge provides an effective ultimate live load strain for the bridge caused by heavy vehicle effects. In the case of the Bridge Manual, an extrapolation out to a 95% confidence limit in 100 years is appropriate to represent an ultimate live load strain. For the serviceability limit state, an extrapolation out to a 95% confidence limit in one year is appropriate. This is also consistent with the AUSTROADS Bridge Design Code (1992).

To allow an assessment of a bridge using Health Monitoring techniques which is consistent with the Bridge Manual requires an integration of the standard equations with Health Monitoring principles.

Re-arranging Equation 1 by moving the Overload Load Factor to the left-hand side gives Equation 5, with $\gamma_o R_o$ representing the capacity available for factored load effects (ultimate live load capacity) imposed by heavy vehicles.

$$\gamma_o R_o = \phi R_i - \gamma_D (DL) - \sum(\gamma(Other\ Effects)) \qquad \text{(Equation 5)}$$

The posting evaluation can then be calculated in terms of ultimate load effects using the ultimate traffic load effect extrapolated from the health monitoring data, rather than the posting load effect, as demonstrated in Equation 6. In this way the bridge's ability to safely carry the actual traffic using the bridge during its design life (based on the traffic during the monitoring period) is calculated. The evaluation that is derived from this procedure has been defined as the Fitness for Purpose Evaluation.

$$FPE = \left(\frac{\gamma_o R_o}{UTL \ \ Effect} \right) \times 100 \ \% \qquad \text{(Equation 6)}$$

where:

FPE = Fitness for Purpose Evaluation

$\gamma_o R_o$ = Ultimate Traffic Live Load Capacity

$UTL\ Effect$ = Ultimate Traffic Load Effect derived from health monitoring data

Generally a Fitness for Purpose Evaluation greater than 100% indicates that the structure is "Fit for Purpose", while an Evaluation of less than 100% indicates that intervention is required. This intervention could include repair, rehabilitation, replacement, risk management, or a load limit.

2.4.2 Behavioural Test using a Known Vehicle

The Health Monitoring approach relies on statistical techniques to provide a rating for bridges. This involves installing an instrumentation system on the bridge. It is often possible, with little extra effort, to record the response of the bridge to several events generated by a heavy vehicle of known mass and configuration (i.e. a known vehicle). This vehicle can be any legally loaded heavy vehicle. It can then be modelled and used as a load case in the analytical model required for a theoretical evaluation. While this activity is technically not required for Health Monitoring, it has a number of benefits. For example, results from the known vehicle can be used to calibrate the health monitoring data. These can provide:

- A mechanistically derived indicator of the extent of overloaded vehicles in the health monitoring data, which can be used to confirm the statistical indicators of the presence of overloading;
- An indication of whether the bridge behaviour is adequately predicted by the analytical model used for evaluation; where there is significant variation, it can provide a general indication of the source of variation;
- Quantification of the dynamic increment that actually exists at the bridge;
- Greater detail of the transport task to which the bridge is subjected.

Behavioural tests using a known vehicle were conducted at the Waipawa Bridge during the Health Monitoring programme, and the results are given in section 4.4 of this report.

3. Bridge Description & Assessment

This section outlines the description of the Waipawa Bridge and its classification based on the guidelines set out in the Bridge Manual. The results of an assessment of the bridge capacity are also presented to determine the predicted mode of failure and identify critical locations for health monitoring instrumentation.

3.1 Bridge Description

The Waipawa Bridge is located on State Highway (SH) 2, and crosses the Waipawa River near the town of Waipawa, about 70 km south of Hastings, in Hawke's Bay Region, North Island. The bridge consists of fourteen simply supported spans 21.6 m in length. Each span includes four steel plate "I" girders supporting a composite reinforced concrete deck. Channel shear connectors connect the deck slab to the steel girders. Construction of the 304 m-long bridge was completed in 1958 and is illustrated in Figure 3.1.

Figure 3.1 Waipawa Bridge, in southern Hawke's Bay, North Island, New Zealand.

The girders of the structure are braced diagonally at the ends and at regular intervals along the length of each girder. This bracing is illustrated in Figure 3.2. The bridge is generally in good condition, and the transverse cracks that occur in the deck slab at regular intervals are believed to be caused by shrinkage effects. The drawings show that the slab is lightly reinforced longitudinally and that this reinforcement is grouped in bands between the girders to avoid the channel shear connectors.

Figure 3.2 Bracing of the main girders.

The current theoretical load rating of the bridge listed in the TNZ Structural Inventory (1999) is:

- Bridge Classification (superstructure) 83%
- Deck Capacity Factor (DCF) 0.6

These ratings are based on the evaluation methods set out in Section 6 of the Bridge Manual, which are outlined in Section 2.3 of this report.

3.2 Structural Assessment

To identify the critical failure modes of the superstructure, a typical span of the bridge was investigated using a "grillage analysis"[1]. The dimensions of the structure were taken from the "as constructed" plans and were confirmed by measurements taken on site.

The actual material properties were not available. The values used were taken from Section 6.3.4 of the Bridge Manual. The following properties (nomenclature as in the Bridge Manual) were adopted for the grillage analysis:

- Steel Girders f_y = 230 MPa, E = 200 000 MPa
- Concrete Deck f'_c = 21 MPa, E = 23 600 MPa
- Deck reinforcement f_y = 250 MPa

[1] Grillage analysis: analytical model using a 2-dimensional idealisation of the bridge superstructure as beam elements.

The grillage analysis was used to determine the response of the structure for the rating load case. The rating load case, as described in the Bridge Manual, consists of the 0.85 HO vehicle and the 0.85 HN vehicle in adjacent lanes with the vehicles positioned at eccentricities to produce the worst-case load effects. The effect of the known vehicle (detailed in section 4.2 of this report) was also analysed.

3.2.1 Girder Bending

The maximum bending moments for each girder for the analysis loads are listed in Table 3.1. The difference in the bending moments for each girder for dead load is caused by the additional load of the guardrails and material weight of the kerb and footpath on the outside girders. The results presented in Table 3.1 have not been factored and they represent the bending moment envelope for each girder.

Table 3.1 Results of grillage analysis for midspan bending moment (kNm) in the girders.

Load	Girder 1 (Footpath)	Girder 2	Girder 3	Girder 4 (Kerb)
Dead Load	1578	872	872	1207
Known Vehicle	481	394	388	554
0.85 HO + 0.85 HN Vehicles (Rating Load)	1060	976	1070	1285

The bending capacity of the main steel girders of the superstructure has been calculated in accordance with the Steel Structures Standard (NZS 3404: Part 1 1997). The girders were found to be only partially composite with the concrete deck. The nominal member moment capacity (ϕM_{bx}) was calculated for the central two girders and the girders under the kerb and the footpath. The capacities are 4088 kNm for the central girders and 4908 kNm for the footpath and kerb girders.

3.2.2 Girder Shear

The maximum shear force for each girder was also found for the analysis loads using the grillage analysis. The results are presented in Table 3.2 and represent the shear envelope for each girder.

The shear capacity (ϕV_{vu}) of the main girders was determined in accordance with the Steel Structures Standard (NZS 3404: Part 1 1997), and is 1377 kN.

Table 3.2 Results of grillage analysis for shear (kN) in the girders.

Load	Girder 1 (Footpath)	Girder 2	Girder 3	Girder 4 (Kerb)
Dead Load	293	161	161	213
Known Vehicle	83	101	81	117
0.85 HO + 0.85 HN Vehicles (Rating Load)	162	218	230	250

3.2.3 Deck Capacity

3.2.3.1 *Plate Bending*

The critical case for bending in the deck was determined using the Deck Rating Loads given in Table 6.7 of the Bridge Manual. The loads include the twin-tyred load for the HN axle and both options of the HO axle loading (Section 3.1.2 in Bridge Manual).

The effective transverse span of the deck was calculated as 2215 mm. This was the mean of the clear span between the flanges of the girders (2070 mm), and the distance between the webs of the girders (2360 mm) as stated in Section 6.5.4 of the Bridge Manual.

Analysis found that the HO wheel rating load was critical with the resulting bending moment being 32 kNm, assuming the deck is continuous over the girders. The effective width of deck slab resisting this footprint was assumed to be 1900 mm and the bending capacity of the deck at the ultimate limit state (ϕM_u) was 54 kNm.

3.2.3.2 *Empirical Method*

The capacity of the deck can also be calculated using empirical methods presented in the Bridge Manual (Section 6.5.2). This capacity is determined from Figures 6.1 to 6.5 in Section 6.5.2 of the Bridge Manual. The DCF is then calculated using Equations 7(a) and 7(b).

$$DCF = \frac{Overload\ Wheel\ Load\ Capacity}{Rating\ Load\ Effect} \qquad \text{(Equation 7a)}$$

$$DCF = \frac{\phi R_i}{\gamma_o * 95 * I} \qquad \text{(Equation 7b)}$$

The allowable wheel loading of the deck was found to be 225 kN. This allowable wheel load gives a DCF of 1.27 which corresponds to a Deck Grade of A. The strength for this method is also dependent on the assessment of cracking in the slab. For this assessment the assumption regarding the deck condition influences the result, and for the Waipawa Bridge the deck condition was assumed to be good.

3.3 Theoretical Load Evaluation

The process required to determine the theoretical load evaluation of a bridge, using the Bridge Manual, is outlined in section 2.2 of this report. The results of the theoretical evaluation for this structure are presented in Tables 3.3 and 3.4. The evaluation has been assessed for bending and shear in both the girders and the deck. The table also presents a comparison of the evaluations calculated by Infratech Systems & Services (Infratech), and the load rating recorded in the current (1999) TNZ Structural Inventory.

A value of 1.25 was used for the impact factor on the main girders, and a value of 1.3 was used for the deck. A dead load factor (γ_D) of 1.3 was also used in the analysis. The kerbside edge girder is the most critical for both shear and bending.

Table 3.3 Summary of theoretical load evaluations for the main girders.

Mode of Failure	ϕ * Ultimate Capacity	0.85 HO Rating Load	Dead Load	HO Rating (Infratech)	Rating (Structural Inventory)
Girder Bending	4908kNm	1285kNm	1207kNm	140%	83%
Girder Shear	1377kN	250kN	213kN	236%	

Table 3.4 Summary of theoretical load evaluations for the deck slab.

Mode of Failure	ϕ * Ultimate Capacity	Deck Rating Load	Dead Load	HO Rating (Infratech)	Rating (Structural Inventory)
Deck Capacity, Plate Bending	54kNm	32kNm	4kNm	0.74	0.6
Deck Capacity, Empirical Method	225kN	95kN	–	1.27	

Some differences were recorded between the calculated rating and the rating obtained from the structural inventory, particularly for the main girders. The differences for the main girders may relate to assumptions regarding the degree of composite action or the inclusion of the kerbs in the strength calculations, while for the deck capacity the differences with the empirical capacity may relate to the assumptions regarding the deck condition. The posting evaluation (0.85 HN) is not listed in Table 3.3, but for the Waipawa Bridge is approximately 170%, based on midspan bending of the main girders.

3.4 Summary

The Waipawa Bridge, in Hawke's Bay Region, was analysed using a grillage analysis to determine the bending moment and shear in the girders of a typical span based on various vehicle loadings.

The bending moment in the girders was found to govern the strength and therefore it determines the rating of the superstructure. The rating calculated in this report for the main girders was significantly higher than the rating in the 1999 TNZ Structural Inventory.

The deck rating (0.74) for this structure, according to the TNZ Structural Inventory, is low and may be dependent on the assumed condition of the deck.

Based on the results from this analysis, the Health Monitoring programme concentrated on determining a Fitness for Purpose Evaluation for the girders based on midspan bending of the girders, and the performance of the deck in bending. Shear strains were also measured in the girders to confirm the performance of the girders in shear.

4. Health Monitoring Programme

The programme of Health Monitoring involved two components:

- Short-term health monitoring of the ambient heavy vehicle traffic for a period of approximately one day.
- Testing using a heavy vehicle of known mass and dimensions (i.e. the known vehicle) to provide a comparison with the health monitoring data.

This section presents the details and results of the Health Monitoring programme on the Waipawa Bridge.

4.1 Instrumentation

The instrumentation installed on the bridge included six Demountable Strain Gauge transducers installed on the girders and deck slab, and two Foil Strain Gauge transducers on the web of the girder. Locations of this instrumentation that was installed on the southern end span of the structure, are illustrated on the plan in Figure 4.1. The instrumentation is also shown on the elevation in Figure 4.2, along with the typical position of vehicles in the lanes on the bridge.

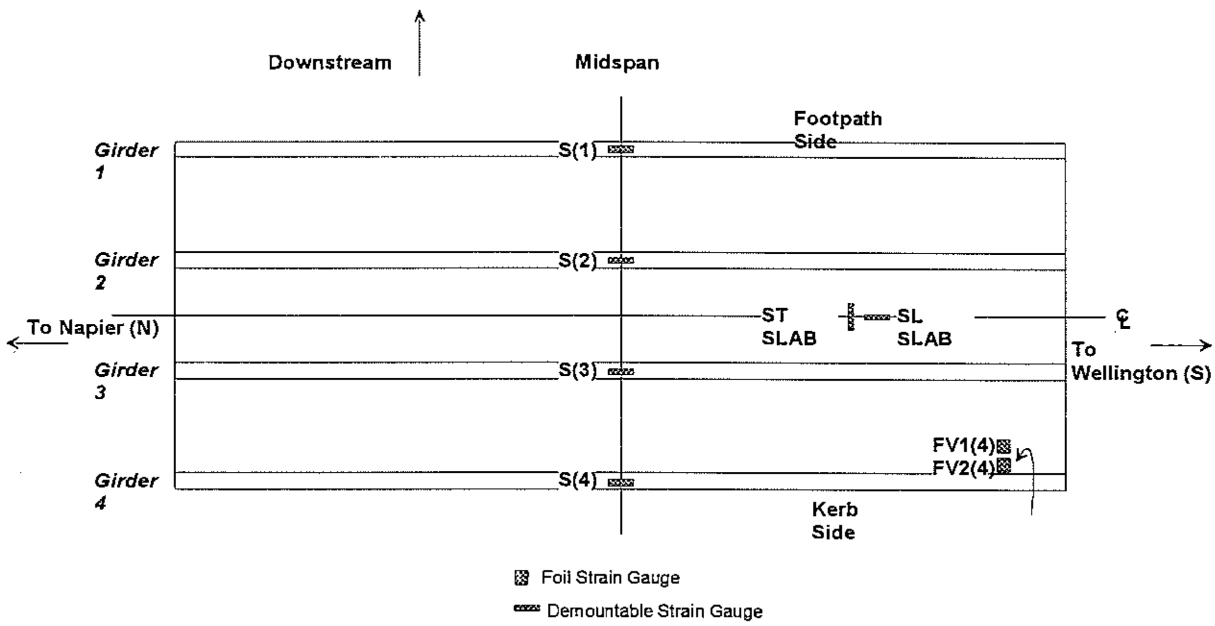


Figure 4.1 Instrumentation plan for the Waipawa Bridge.

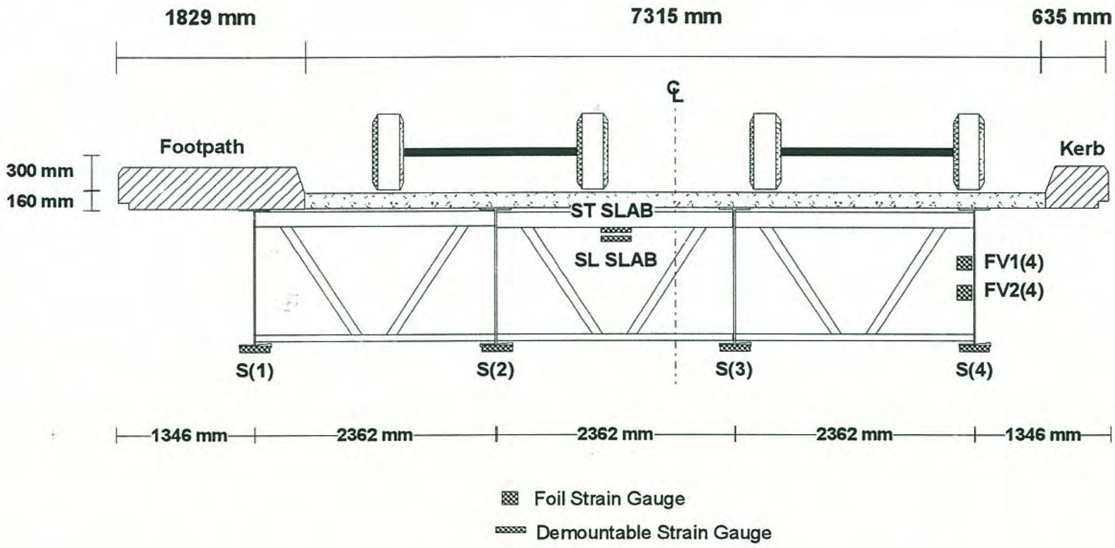


Figure 4.2 **Cross section of structure showing instrumentation and vehicle lane positions.**

Figure 4.3 **Instrumentation installed on the midspan of a typical girder on the Waipawa Bridge.**

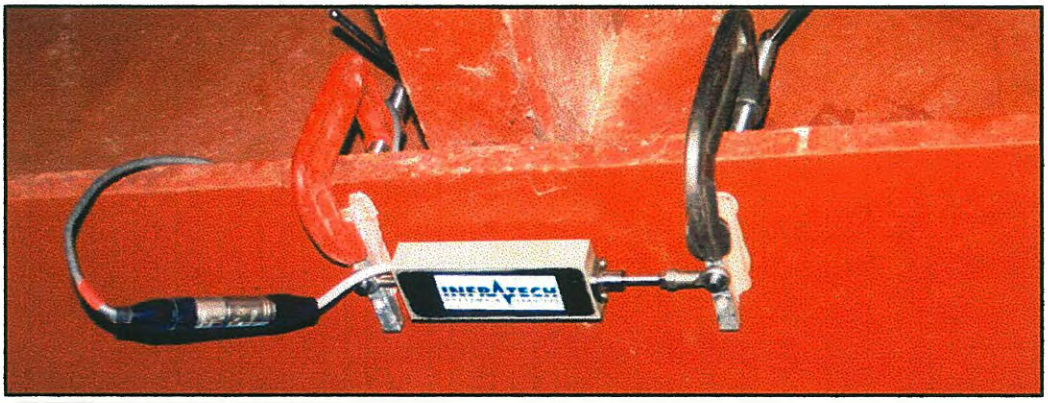


Figure 4.4 **Instrumentation on the deck.**

Figure 4.5 Orientation and position of foil strain gauge transducers.

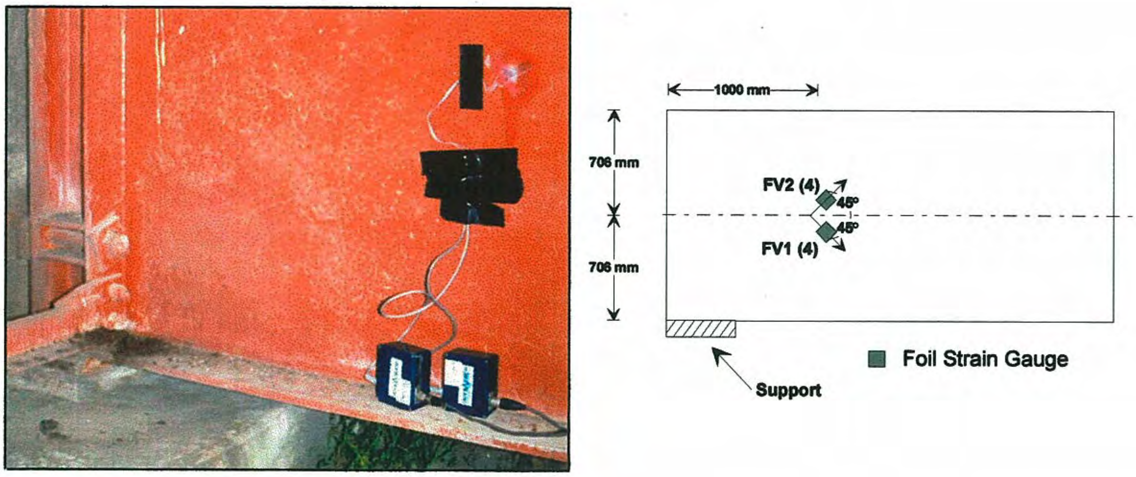


Figure 4.3 illustrates a demountable strain gauge on the midspan of a typical girder. The transducers installed on the concrete deck slab are illustrated in Figure 4.4, and they were installed to measure the transverse and longitudinal bending in the deck. The transducer measuring the longitudinal strain is located over one of the transverse shrinkage cracks. The orientation and position of the foil strain gauges is illustrated in Figure 4.7. These gauges were installed on the southern end of Girder 4 to measure the shear response of the girder.

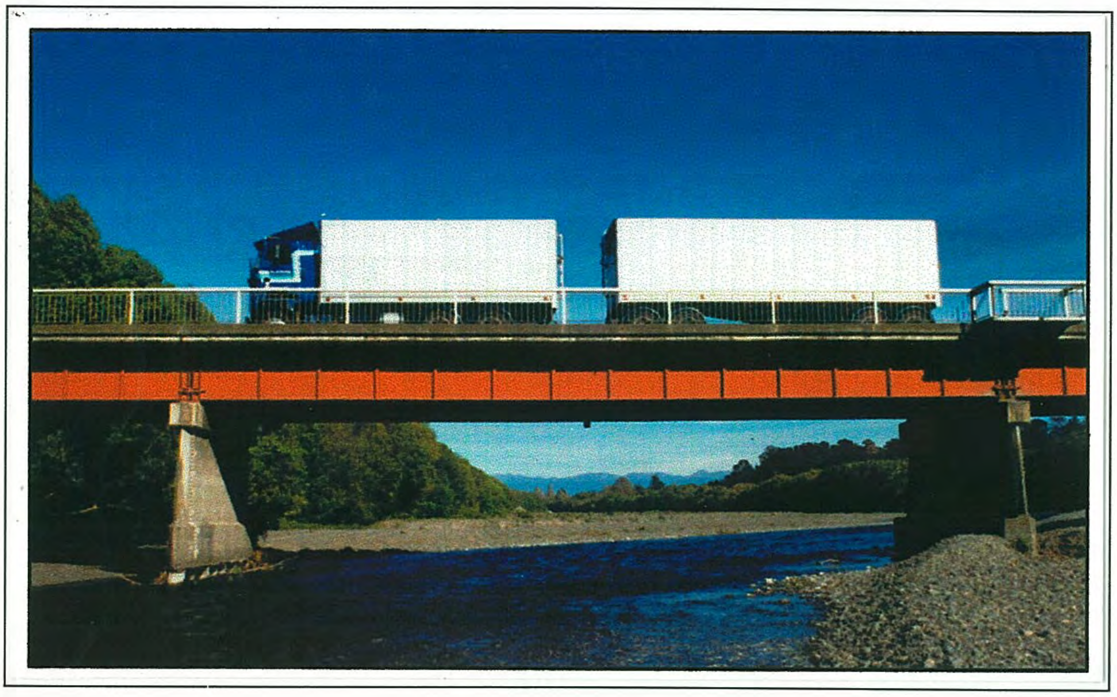
The demountable strain gauges (gauge length 230 mm) used on the girders measured strain at a point 20 mm below the soffit of the girders and slab. To account for this, the data presented in this report have been adjusted according to the size of the section in each case. The sign conventions used throughout this report include positive values for tension strains and negative values for compressive strains.

4.2 Procedure

The health monitoring of the structure began on Wednesday 7 October, and continued until Thursday 8 October, 1998, giving a total monitoring period of approximately 22 hours. During the one-day monitoring period, the response of the bridge to 422 heavy vehicles was recorded, excluding the passage of the known vehicle.

In order to provide a control for all the data gathered during the entire monitoring period, the behaviour of the bridge in response to a known load (i.e. a heavy vehicle of known mass and dimensions) was measured. This component of the Health Monitoring programme was conducted on Thursday 8 October, 1998. The vehicle used for the testing was supplied by Higgins Contractors Ltd, Palmerston North (shown on the bridge in Figure 4.6). It was a seven-axled heavy vehicle of known gross mass of 45.62 tonnes, and with the dimensions illustrated in Figure 4.7.

Figure 4.6 The known vehicle used for behavioural testing on the Waipawa Bridge.



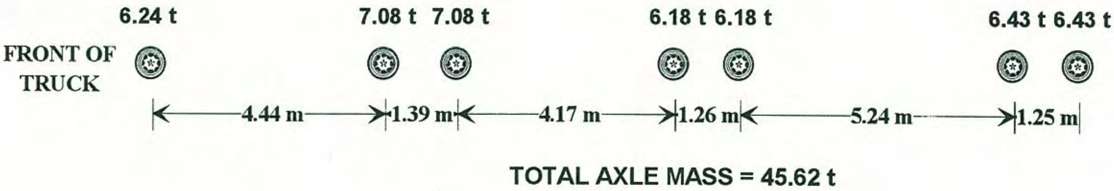


Figure 4.7 Axle mass and configuration of the known vehicle.

The testing with the known vehicle was conducted by recording the response of the transducers on the bridge to the vehicle as it passed over the structure at different speeds, that ranged from 20 km/h to 80 km/h, in increments of 10 km/h. In each case the truck was positioned in the normal traffic lane.

Testing was completed by slowing the traffic in each direction, or in some cases stopping it for several minutes at a time. This ensured that minimal traffic interruptions were experienced and also allowed continuous monitoring of ambient heavy vehicles between test runs with the known vehicle.

4.3 Short-Term Health Monitoring Results

4.3.1 Girder Response

A typical strain response versus time was graphed (as waveforms, in Figure 4.8), for the midspan bending strains recorded during the passage of a typical heavy vehicle. The responses of Girders 3 and 4 are very similar, indicating that the vehicle passed directly over these girders. The dynamic activity of the girders after the vehicle has passed over the instrumented span is obvious. This response is typically from one of the larger events recorded.

Some continuity exists between the instrumented span and the adjacent span. This is evident by the small negative (compression) strains, illustrated in Figure 4.8, recorded after the vehicle had passed over the instrumented span heading north.

The waveforms for the transducers that measured the shear response in Girder 4 are presented in Figure 4.9. Both transducers experienced a similar response in the opposite sense. Transducer FV1(4) measured tensile strain while FV2(4) measured compressive strain.

The response of each girder for every recorded event is best presented on a scatter diagram. The scatter diagrams represent the extreme values recorded for each event during the Health Monitoring period The results from the known vehicle are not included in Figures 4.10 and 4.11. These diagrams give an indication of the mass and distribution with time of the heavy vehicles using this route, and Figure 4.10 presents the scatter diagram for the midspan transducers.

Figure 4.11 presents the scatter diagram for the transducers measuring the shear response in Girder 4.

The data from the scatter diagram can also be plotted on a histogram that incorporates a cumulative distribution. The example presented in Figure 4.12 is for transducer S(2) only.

Figure 4.8 Waveforms for midspan transducers recorded during the passage of a northbound vehicle.

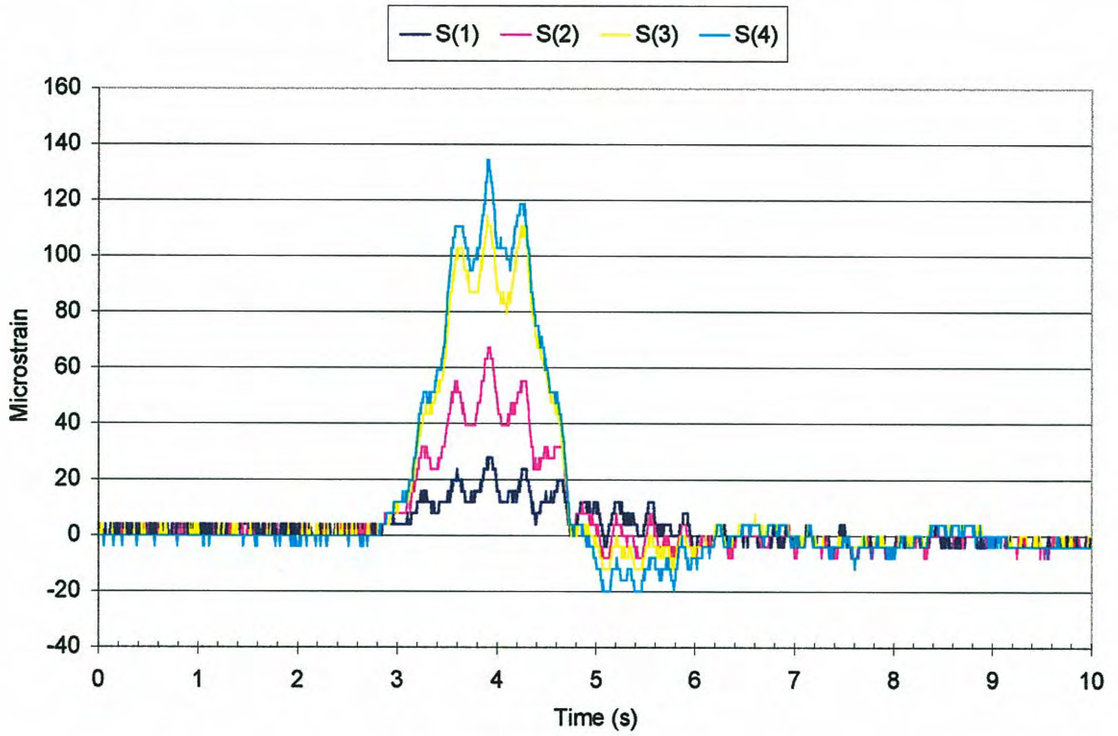


Figure 4.9 Waveforms for shear transducers recorded during the passage of a northbound vehicle.

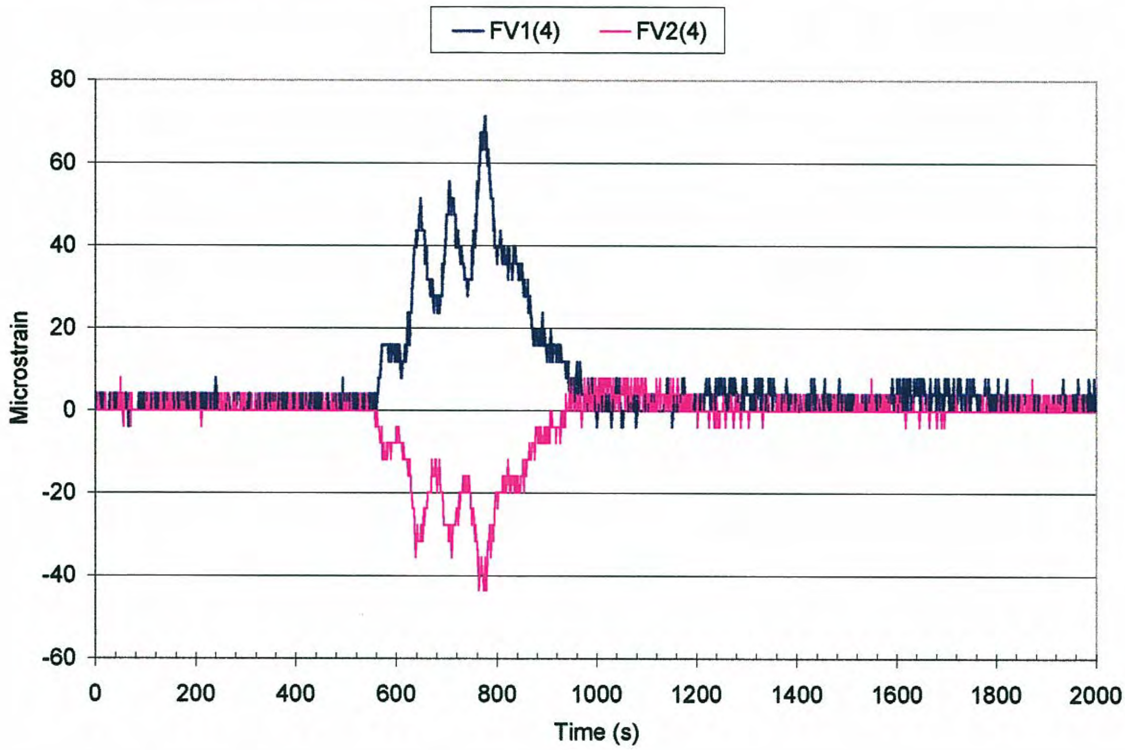


Figure 4.10 Scatter diagram for the midspan transducers.

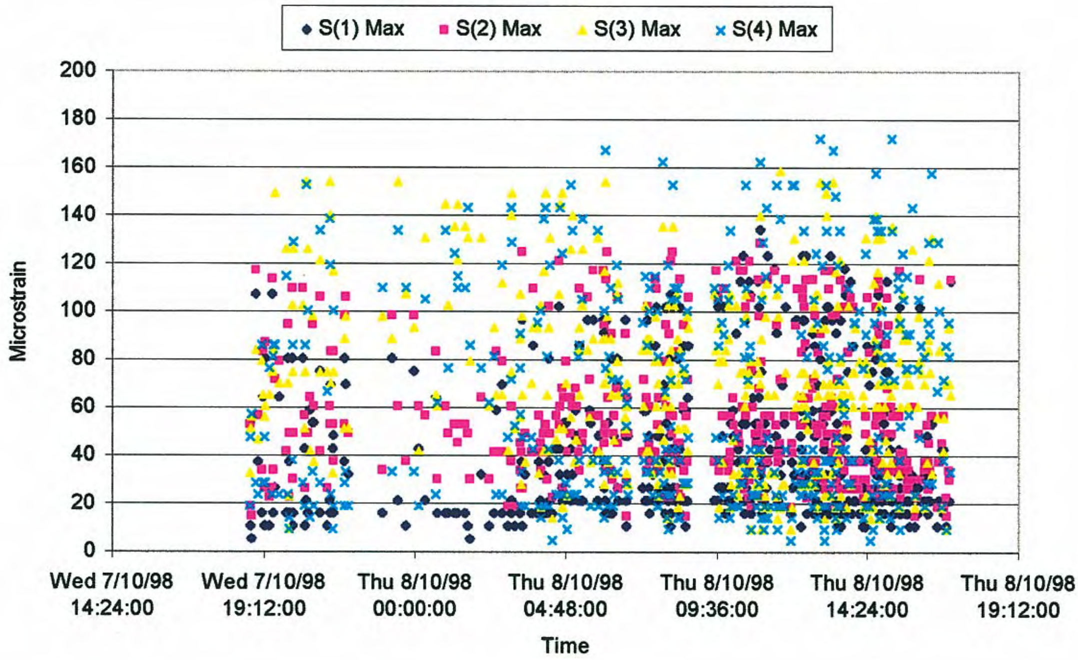


Figure 4.11 Scatter diagram for the shear transducers.

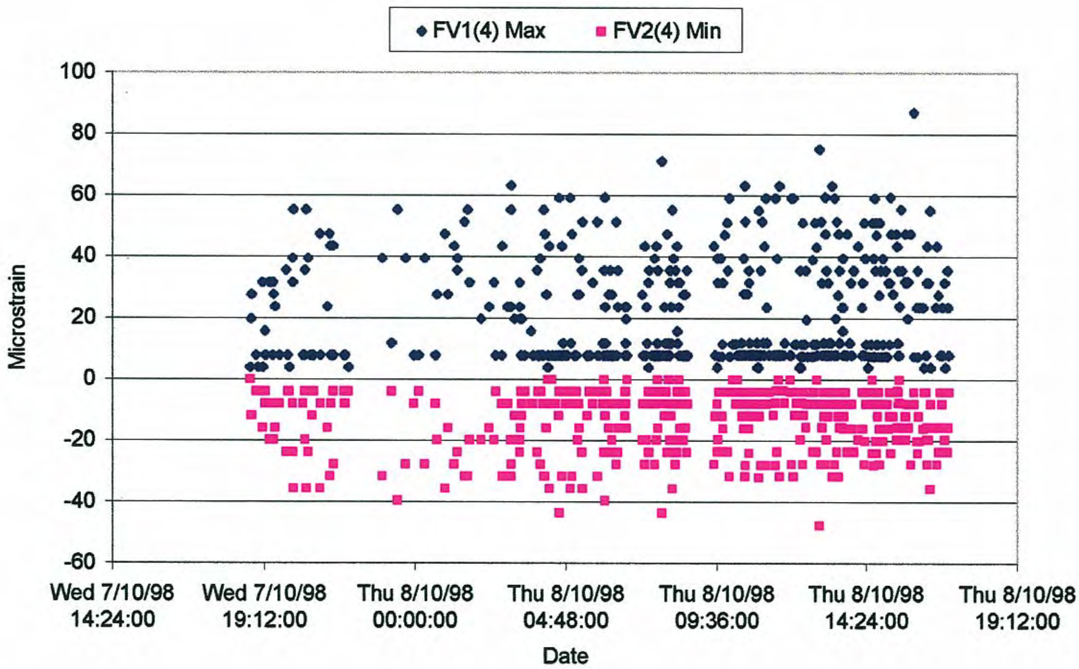
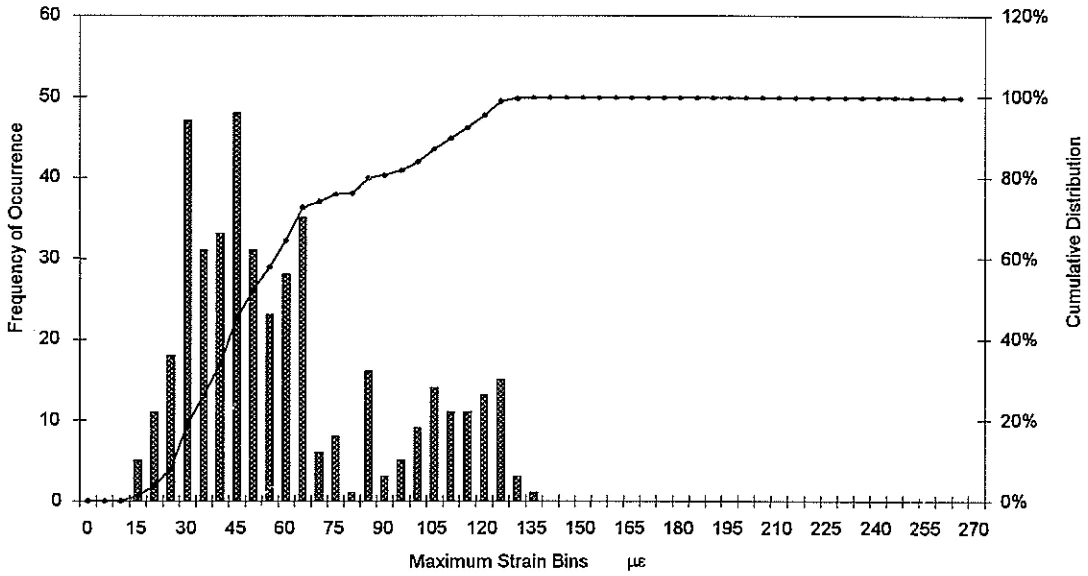


Figure 4.12 Histogram and cumulative distribution function for midspan transducer S(2).

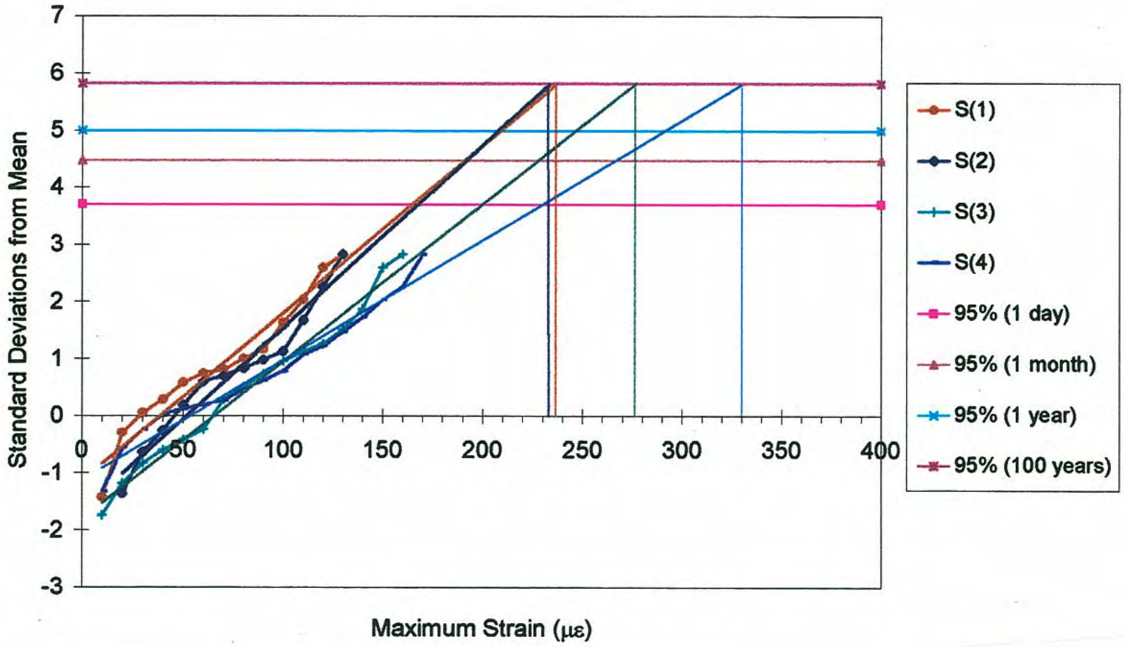


The cumulative distribution function can then be plotted on a probability scale known as an "inverse normal scale". The inverse normal plot for each of the midspan transducers is presented in Figure 4.13. On this graph the vertical scale represents the number of standard deviations that each point is away from the mean. The horizontal scale is the maximum strain recorded for each event. The point at which the data plot crosses the horizontal axis represents the average (mean) strain, and a straight line represents a normally distributed sample of data.

Horizontal lines representing the expected position of the 95% confidence limit for the data for 1 day, 1 month, 1 year, and 100 years have been plotted. Extrapolating the recorded data allows estimates of strain for these longer return intervals. The strain extrapolated for the 95% confidence limit for 100 years represents the ultimate traffic load effect for the Fitness for Purpose Evaluation, as outlined in section 2.4 of this report.

The data in Figure 4.13 indicate that the extrapolated strain estimated for Girder 1 (under the footpath) was approximately 225 με, and the strain in Girder 2 is similar to that in Girder 1. The strain in Girder 3 was approximately 260 με, and the highest strains (330 με) were recorded in Girder 4.

Figure 4.13 Inverse normal plot for midspan transducers.



The distributions for the midspan transducers are not linear, indicating that the data are not normally distributed. A closer inspection of the scatter diagram (particularly for transducer S(2), in Figure 4.10) shows that the data have two distinct populations. This is better illustrated on the histogram in Figure 4.12. The reasons for these two populations may be two different groups of heavy vehicle traffic, or the effects of vehicles travelling in different directions (lanes) on the bridge.

Figures 4.14 and 4.15 show inverse normal plots for the transducers with the data separated into directions. Figure 4.14 illustrates the data for the vehicles travelling south, and Figure 4.15 shows the data for the vehicles travelling north. These plots show that the data for transducers S(3) and S(4) are normally distributed. However the data for transducers S(1) and S(2) are not normally distributed. This may be related to different vehicle types, or to some effect associated with the positioning of vehicles on the bridge and the influence on load distribution into the girders.

An extrapolated strain for S(4) of 330 με has been used in this investigation to ensure that stringer 4 is evaluated on the same basis as the other stringers.

Figure 4.14 Revised inverse normal distributions (for midspan transducers) for vehicles travelling south.

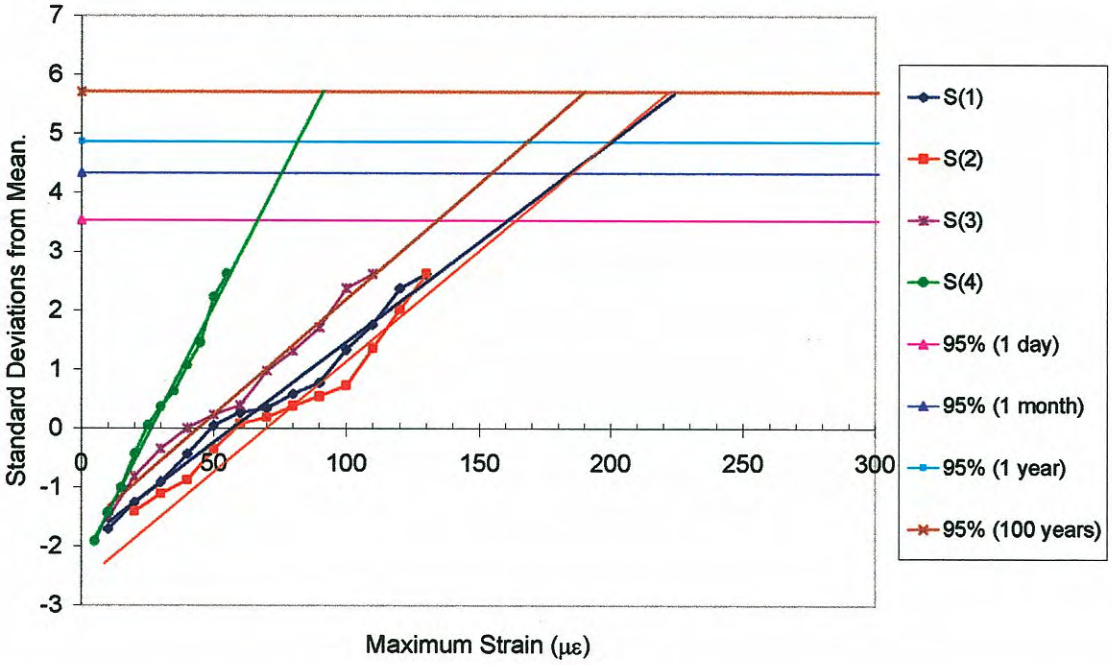
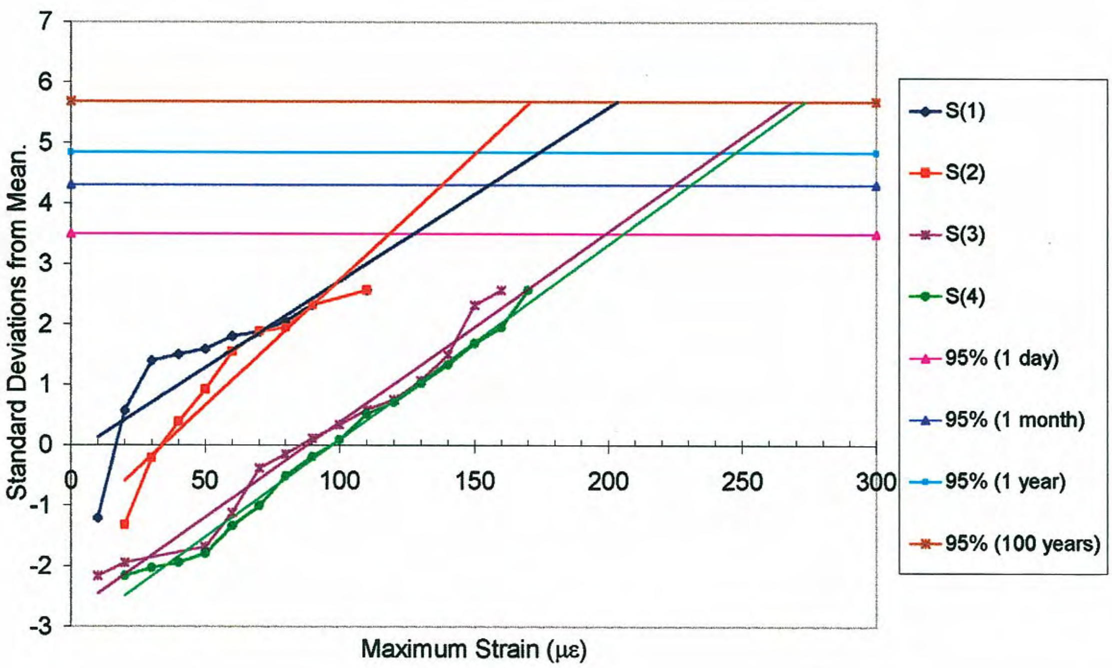
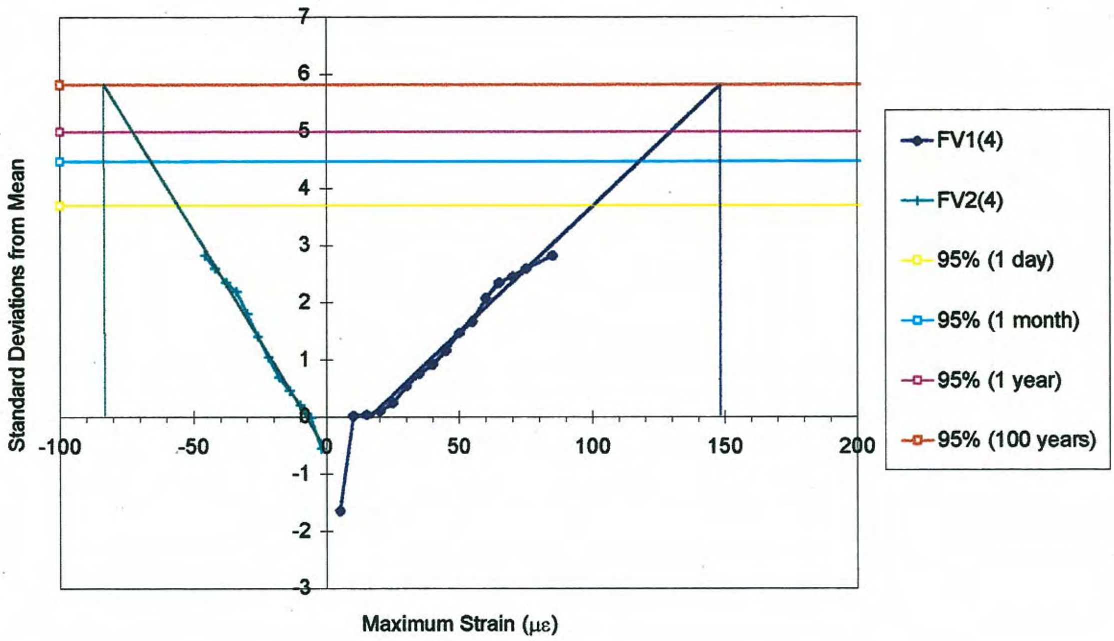


Figure 4.15 Revised inverse normal distributions (for midspan transducers) for vehicles travelling north.



The inverse normal distributions for the foil strain gauges measuring the shear effects in Girder 4 are presented in Figure 4.16.

Figure 4.16 Inverse normal distribution for foil strain gauges.



The maximum recorded strains from the Health Monitoring, along with the extrapolated results, are presented in Table 4.1 for the midspan transducers and the transducers measuring the shear effects. The extrapolated data for the midspan transducers are based on the inverse normal plots for all data.

Table 4.1 Extrapolated data from inverse normal distributions.

Transducer	Maximum Recorded Value (Health Monitoring)	Extrapolated Value (95% Confidence limit for 1 year)	Extrapolated Value (95% Confidence limit for 100 years)
	Strain ($\mu\varepsilon$)		
S(1)	135	200	225
S(2)	130	205	230
S(3)	160	235	260
S(4)	170	245	330
FV1(4)	87	130	150
FV2(4)	-50	-70	-80

4.3.2 Deck Response

Figure 4.17 shows clearly the response of the deck to each individual axle as it passed over the instrumented span. The waveforms show differences between transverse and longitudinal bending strains in the deck.

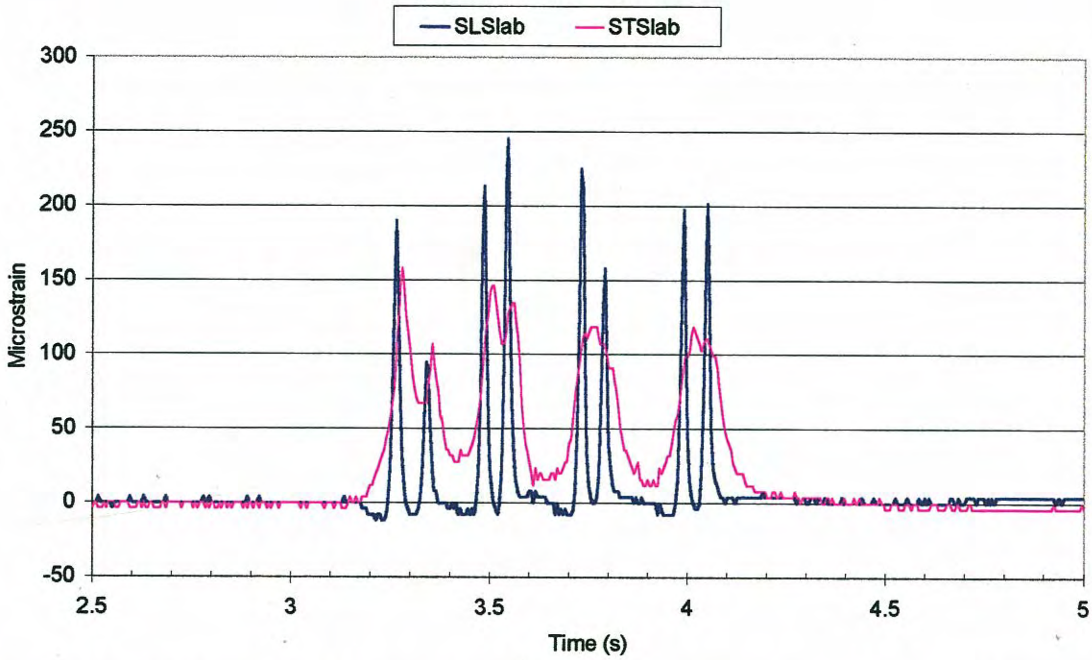


Figure 4.17 Waveform for deck transducers for the passage of a northbound vehicle.

The scatter diagram for the deck transducers is presented in Figure 4.18. The maximum event recorded for transducers SLSlab (longitudinal) and StSlab (transverse) were 330 με and 165 με respectively. The scatter diagram shows that longitudinal deck strain was consistently higher than the transverse deck bending strain. This is because the transducer that measured the longitudinal strain was located over one of the transverse cracks in the slab. Figure 4.19 presents the inverse normal distribution for the deck transducers, and the extrapolated values for the two deck transducers are summarised in Table 4.2.

Table 4.2 Summary of extrapolated events for deck transducers.

Transducer	Maximum Recorded Value (Health Monitoring)	Extrapolated Event (95% confidence limit for 1 year)	Extrapolated Event (95% confidence limit for 100 years)
	Strain (με)		
SLSlab	330	480	545
StSlab	165	270	310

Figure 4.18 Scatter diagram for transducers installed on the deck.

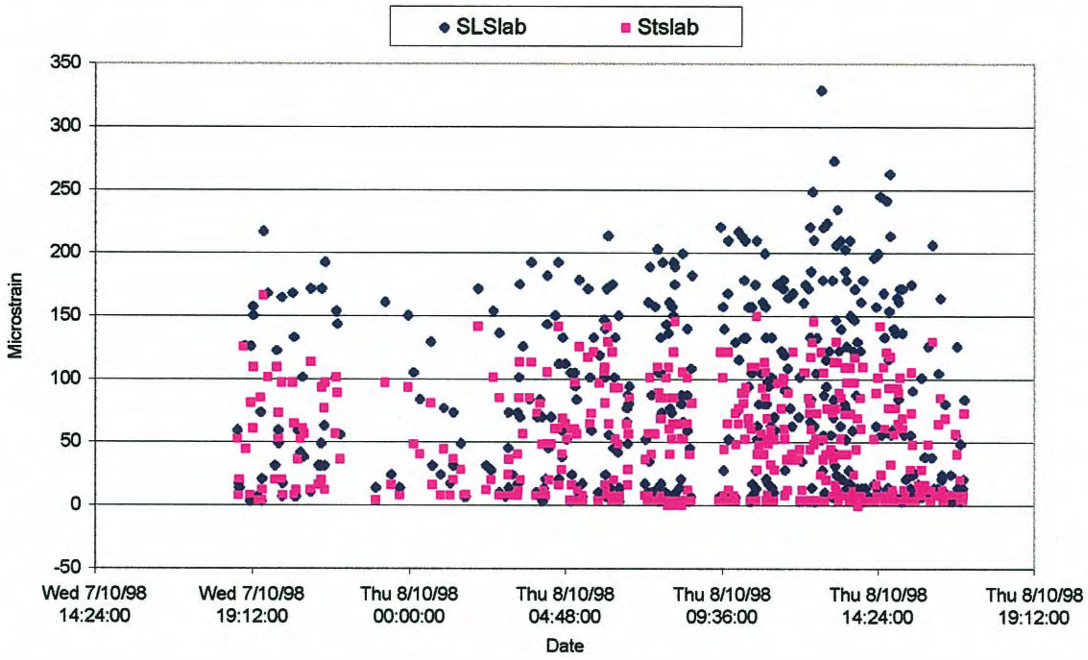
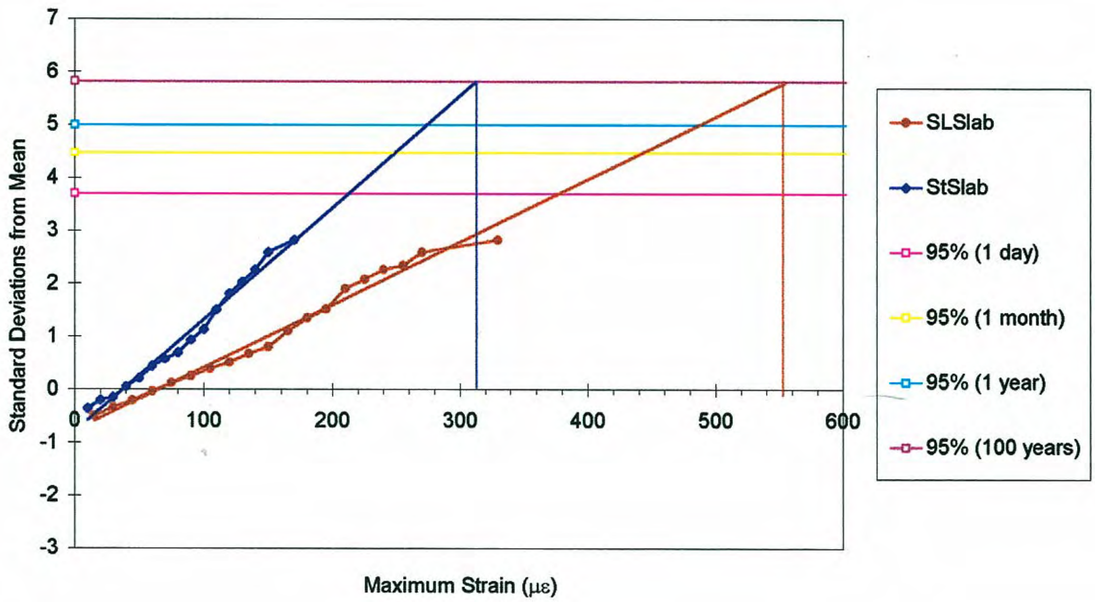


Figure 4.19 Inverse normal distribution for transducers installed on the deck.



4.4 Known Vehicle Testing

4.4.1 Girder Response

A typical waveform from the testing with the known vehicle is presented in Figure 4.20. In this figure, tensile strains are positive and compressive strains are negative.

Figure 4.20 Waveforms for midspan transducers for northbound vehicle travelling at 80 km/h.

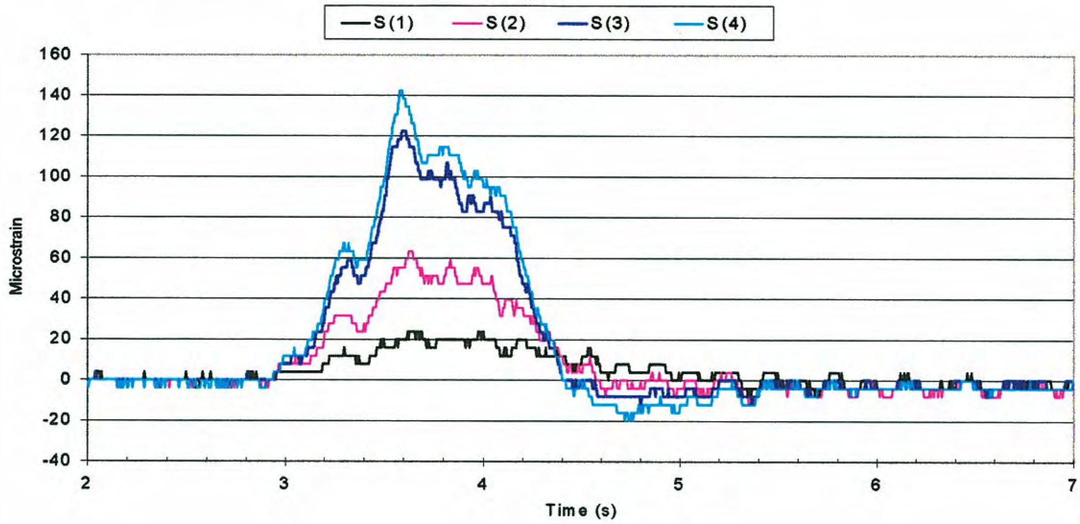
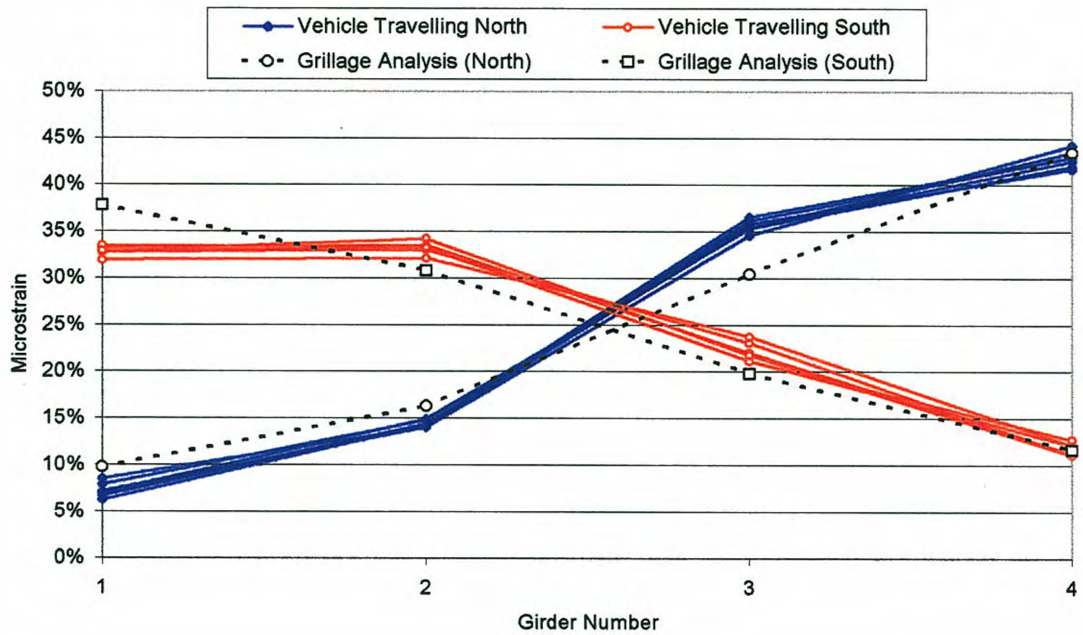


Figure 4.21 Distribution factors for behavioural testing with known vehicle.



The dynamic response of the structure after the vehicle passed over the span is evident. The compressive (negative) strains recorded after the vehicle had passed over the span indicate some continuity between the instrumented span and its adjacent span, which is described in section 4.3. The maximum strains from the testing for each of the transducers on the main girders are presented in Table 4.3.

Table 4.3 Maximum strains recorded for testing with the known vehicle.

Transducer	Maximum Strain (με)
S(1)	125
S(2)	125
S(3)	145
S(4)	170
FV1(4)	70
FV2(4)	–40

The distribution of strain into each girder for the known vehicle testing is presented in Figure 4.21. The distributions have been separated into directions and show consistent results for each test. The distribution of load, based on the grillage analysis, for the known vehicle is also presented. Generally the distributions compare well. There are some differences that are probably related to variations in vehicle position and the higher stiffness in the edge girders. The variations related to vehicle position are because the analytical distribution is based on positioning the vehicle close to the kerb (maximum eccentricity), rather than based on the actual lane position of the vehicles on the bridge.

The dynamics of the main girders of the structure are illustrated by the small oscillations of the span after the vehicle had passed (Figure 4.20). The free vibration response can be considered as the natural frequency of the span, and this is approximately equal to 4 Hz. Because the dynamic response is small, the level of damping could not be determined. Higher resolution settings on the monitor may have given a better signal that would allow the damping to be determined.

The dynamic increment is used to indicate the increase in the effect of a vehicle on a structure as the speed increases. The dynamic increment (impact factor) (AUSTROADS 1992) was calculated using the following equation:

$$DI = \frac{\varepsilon_{dynamic} - \varepsilon_{static}}{\varepsilon_{static}} \qquad \text{(Equation 7)}$$

The response of the crawl test was used for the static result in the calculation of dynamic increment. Figure 4.22 presents the dynamic increments for the transducers on the side of the bridge with the known vehicle travelling north (to Napier). Transducer S(1) has not been included in the graph because it was located on the opposite side of the bridge to the path of the vehicle. Similarly the dynamic increments for the transducers on the side of the bridge for the known vehicle travelling south (to Wellington) are presented in Figure 4.23.

These two figures presenting the dynamic increment data show varied responses for each direction. The dynamic increment for the vehicle travelling north is the highest for Girder 2 with a magnitude of 23%. However, when the vehicle is travelling south, the highest dynamic increment is approximately 13% for Girder 3. The differences in dynamic increment between vehicles travelling north and south probably related to the effect of the road profile, because there is a bump at the approach to this bridge which affects vehicles travelling north. Despite the differences in the magnitude of the dynamic increment, the maximum value of 23% for Girder 2 should be used. This compares well with the value of 1.25 for the impact factor, recommended by the Bridge Manual for this bridge.

4.4.2 Deck Response

Figure 4.24 illustrates the response of transducers SLSlab (longitudinal strain) and StSlab (transverse strain) as the vehicle passes over the instrumented span. The waveform for transducer SLSlab shows seven distinctive responses that represent each of the seven axles as they passed over the deck. The maximum response from this waveform is 70 $\mu\varepsilon$. The waveform also shows no prolonged dynamic effects after the passage of the vehicle, as expected because of the stiffness of the slab.

The maximum results for transducers SLSlab and StSlab for the testing with the known vehicle are 135 $\mu\varepsilon$ and 110 $\mu\varepsilon$ respectively.

4.5 Summary

Table 4.4 presents a summary of the data from the Health Monitoring programme and the testing with the known vehicle. The extrapolated values for all transducers are also included.

The maximum values recorded for the known vehicle compare well with the maximum values recorded during the Health Monitoring from the ambient traffic for bending in the main girders. The recorded values for the shear transducers (FV1, FV2) are higher for the ambient heavy vehicle traffic. This may indicate that vehicles inducing higher shear effects than the known vehicle may be in the traffic stream.

Figure 4.22 Dynamic increment versus speed for the known vehicle travelling north.

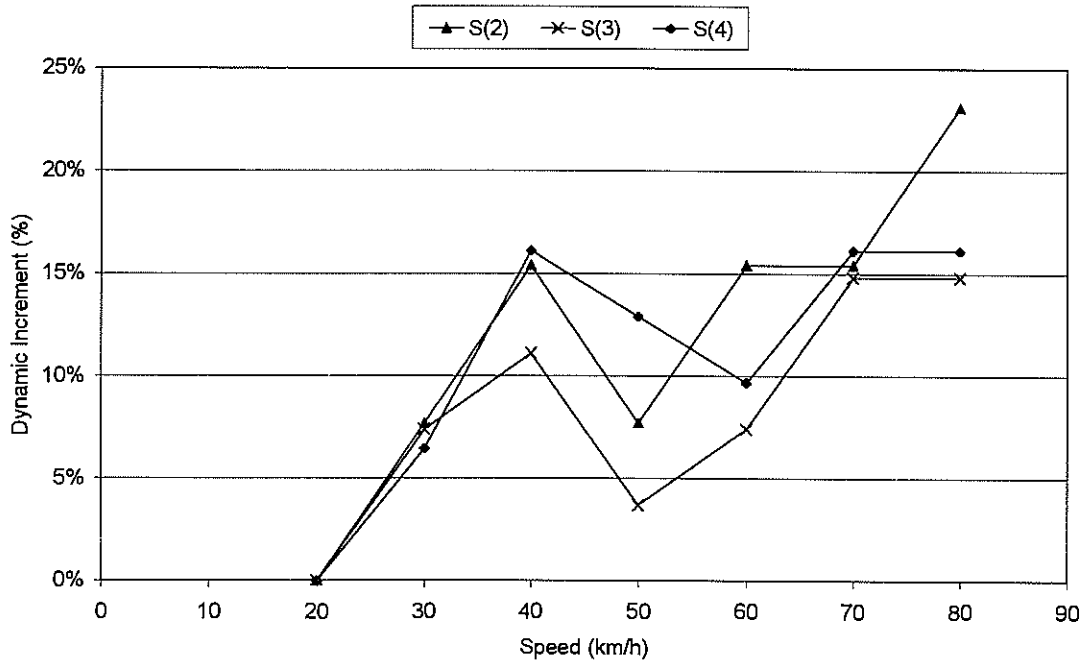
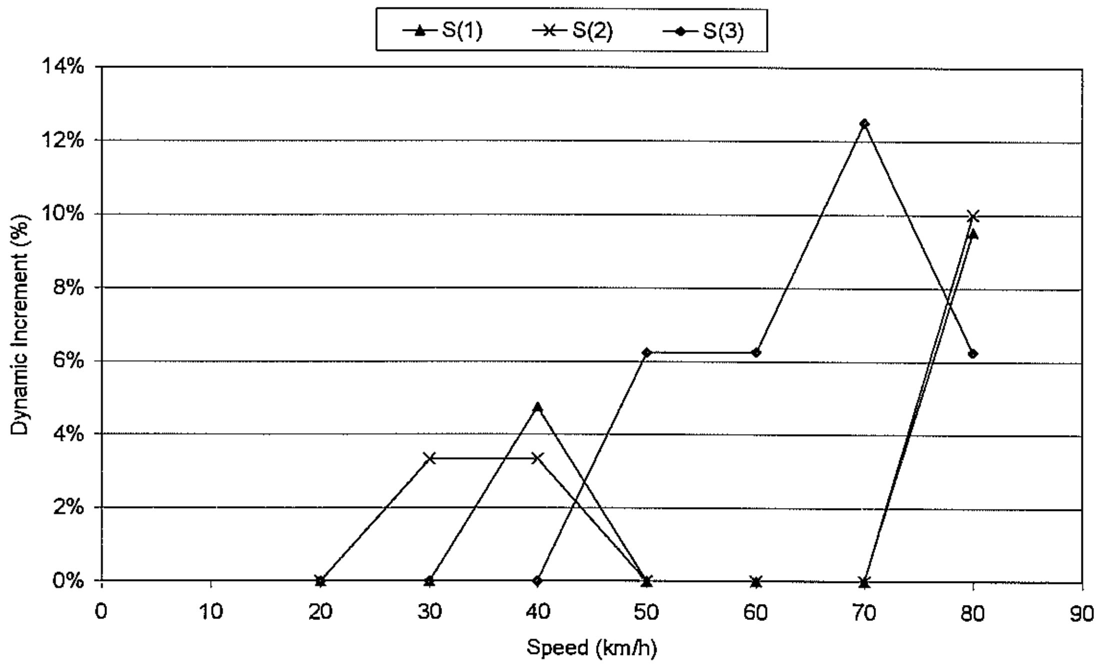


Figure 4.23 Dynamic increment versus speed for the known vehicle travelling south.



The values for the bending in the slab are also much larger for the Health Monitoring. Strains in the deck are more sensitive to wheel positions and a greater range of wheel positions would be expected from the ambient traffic.

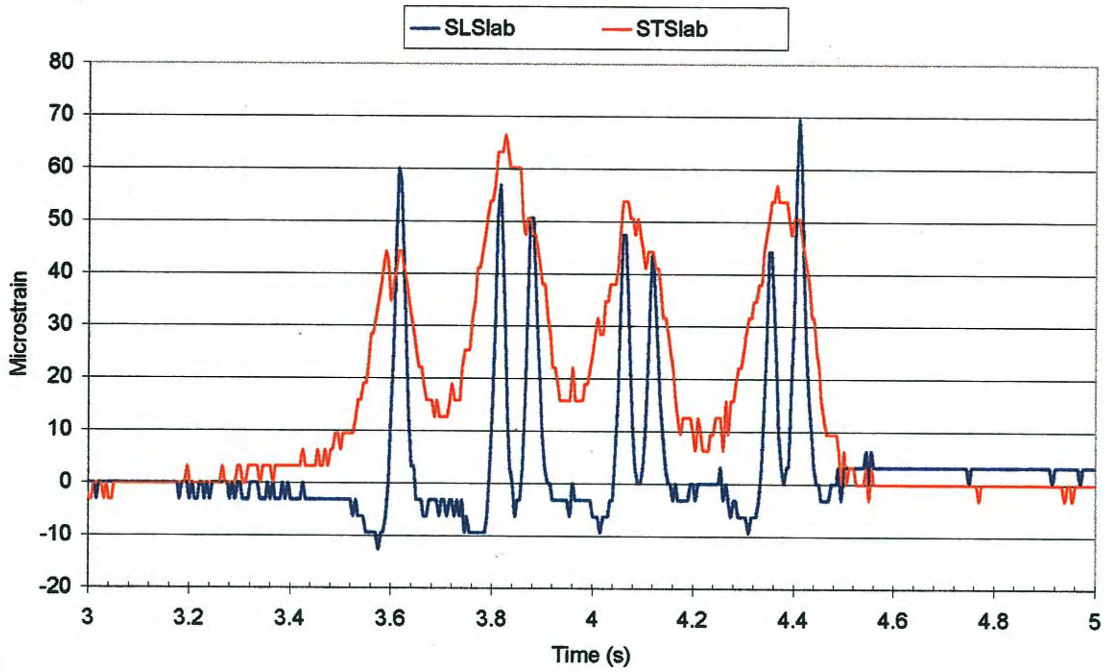


Figure 4.24 Waveform for deck transducers for vehicle travelling south at 80 km/h.

Some vehicles in the traffic stream may also have higher individual axle loads than that of the known vehicle.

The comparison between the results from the Health Monitoring and the known vehicle indicates that the ambient traffic sample is significant and adequate for use in extrapolating the data for the ultimate traffic load effect.

Table 4.4 Summary of health monitoring data.

Transducer	Maximum Recorded Value (Known Vehicle)	Maximum Recorded Value (Health Monitoring)	Extrapolated Value (95% confidence limit) for 1 year	Extrapolated Value (95% confidence limit) for 100 years
	Strain (με)			
S(1)	125	135	200	225
S(2)	125	130	205	230
S(3)	145	160	235	260
S(4)	170	170	245	270
FV1(4)	70	90	130	150
FV2(4)	–40	–50	–70	–80
SLSlab	135	330	480	545
StSlab	110	165	270	310

5. Fitness for Purpose Evaluation

5.1 Steel Girder Bending

The analysis in section 3.2 of this report indicated that midspan girder bending was the critical mode of failure for the girders of the Waipawa Bridge. Thus the Fitness for Purpose has been determined based on this failure mode. The moment capacity (ϕM_{bx} – 1.3DL) for the ultimate traffic live load was 3340 kNm for the critical edge girder and 2950 kNm for a middle girder. These moments correspond to strains in the soffit of the girders of 755 με and 700 με respectively. Figure 5.1 illustrates the extrapolated health monitoring data with these ultimate live load strains superimposed on the graph. This information shows that the ultimate traffic live load strain (95% in 100 years) is approximately one third of the available live load capacity.

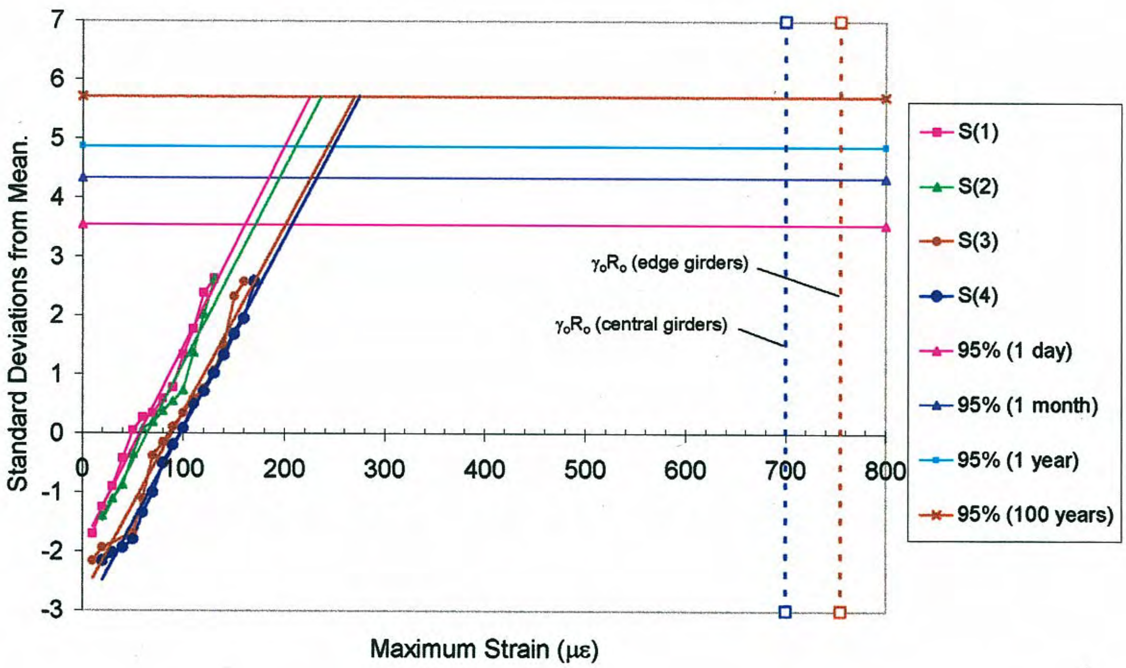


Figure 5.1 Inverse normal distributions for midspan transducers showing the ultimate live load capacity.

The Waipawa Bridge carries two lanes of traffic and therefore the effects of more than one vehicle being on the bridge at any one time must be considered (Multiple Presence). The probability of this occurring on a span of the bridge during the monitoring period is small, and therefore it is expected that a multiple presence event would not have occurred during the monitoring period.

To account for multiple presence events, a number of approaches are available. One is to simulate a multiple presence event by summing the 95% in 100 year event for both lanes. This is consistent with the Bridge Manual method and has been used in this report. The method may be conservative because it assumes that a maximum event occurs in each lane at the same time.

An approach based on Turkstra's Rule (Turkstra & Madsen 1980) may be more appropriate. This rule suggests that an extreme event should be combined only with an average event. In applying the Health Monitoring procedure this means that a maximum event in one lane should be combined with an average event in the other lane. This approach to multiple presence will be confirmed using the long-term monitoring of the Atiamuri Bridge over the Waikato River, another bridge which is part of this project.

Figure 5.2 summarises an assessment of the multiple presence effects on the Waipawa Bridge based on the health monitoring data, and using the method that is consistent with the Bridge Manual. The diagram shows a transverse distribution of strain for each direction and the sum of these two distributions, for the 95% in 100 year event. These distributions are based on the distribution factors from the known vehicle and the extrapolated health monitoring data. The available live load capacity for an edge girder and a middle girder is also shown. These data show that about 54% of the girder capacity would be utilised for a multiple presence event.

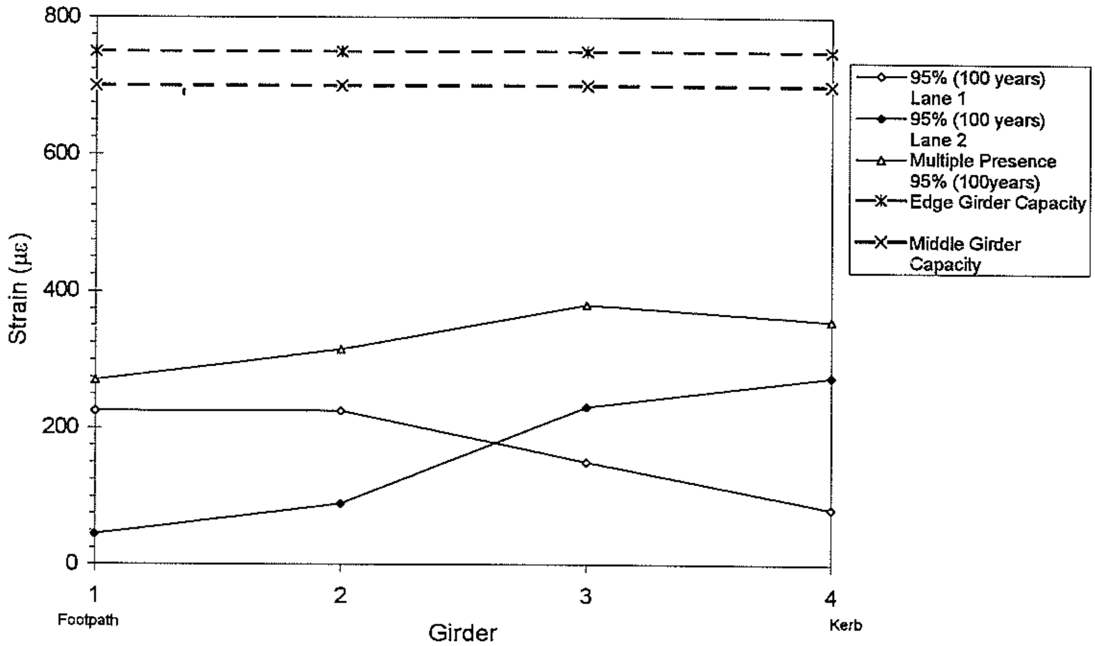


Figure 5.2 Multiple presence strains based on Turkstra's Rule.

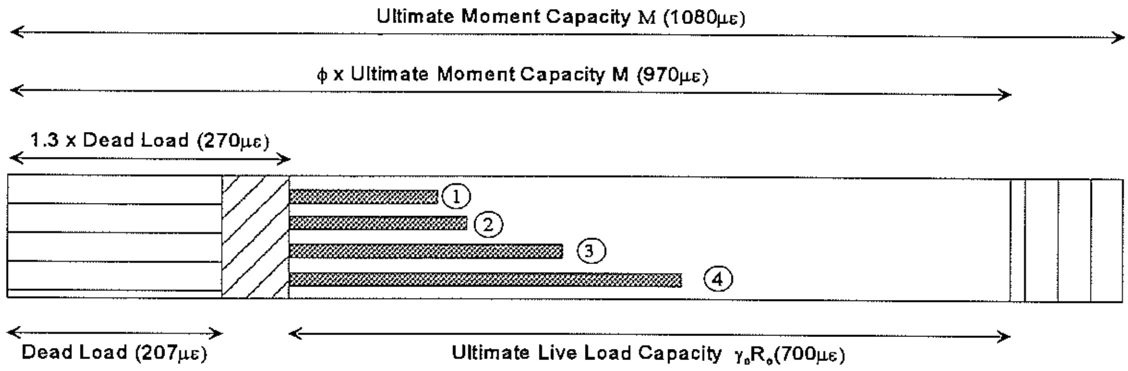
Table 5.1 summarises the calculation of the Fitness for Purpose Evaluation based on the data in Figure 5.2 for bending of the main girders. The method for the calculation of this evaluation was outlined in section 2.4 of this report, and involves dividing the ultimate live load capacity strain by the ultimate traffic load effect determined from the health monitoring data. The Fitness for Purpose Evaluation for this bridge is 180% for the middle girders, which are more critical than the edge girders (with an Evaluation of 210%).

A summary of these results is presented on the capacity diagram in Figure 5.3 for the middle girder. The overall rectangle represents the ultimate moment capacity in bending (1080 $\mu\varepsilon$). This ultimate bending strain does not correspond to the yield strain of the steel because of the effect of the partial composite action.

The diagonally hatched section represents the factored (1.3 times) dead load component (total strain of 270 $\mu\varepsilon$). The white unshaded area is the available live load capacity of the structure available at the ultimate limit state (700 $\mu\varepsilon$). The darkened rectangles represent the magnitude of the recorded strains from the Health Monitoring and testing with the known vehicle, and the extrapolated strains for the ultimate traffic load effect.

Table 5.1 Summary of Fitness for Purpose Evaluation based on the ultimate moment capacity of the middle girder.

Item	Edge Girder	Middle Girder
Strength (ϕM_{bx}) (kNm)	4908	4088
Dead Load (kNm)	1207	872
Ultimate Live Load Capacity Moment ($\gamma_o R_o$) (kNm)	3340	2950
Ultimate Live Load Capacity Strain ($\gamma_o R_o$) ($\mu\varepsilon$)	750	700
Maximum Recorded Strain (Health Monitoring) ($\mu\varepsilon$)	160	170
Known Vehicle Strain ($\mu\varepsilon$)	170	145
Ultimate Traffic Load Effect – Multiple Presence ($\mu\varepsilon$) (95% in 100 years)	360	380
Fitness for Purpose Evaluation (%)	210	180



1. Maximum Recorded Strain - Test Vehicle (145με)
2. Maximum Recorded Strain - Health Monitoring (170με)
3. Maximum Recorded Strain Extrapolated to 95% confidence limit in 100 years (260με)
4. Ultimate Traffic Load Effect (Multiple Presence 95% confidence limit in 100 years 380με)

Figure 5.3 Summary of Fitness for Purpose, based on limit state design principles.

5.2 Steel Girder Shear

The theoretical calculations showed that the shear in the steel girders was not critical (Class 236%). The results from monitoring strains on the web of Girder 4 indicate that the web is behaving similarly to the theoretical behaviour and therefore the web is not limiting the capacity of the structure.

5.3 Concrete Deck

As outlined in section 3 of the report the deck is cracked transversely. These cracks are widely spaced (several metres) and are believed to be caused by shrinkage effects. The strain was recorded across these cracks and, as expected, these strains are relatively high. However the slab is unlikely to fail in a mode in which this cracking would significantly contribute to the failure.

The principal mode of failure for the slab is transverse bending. The transverse bending capacity of the slab (ϕM) is 54 kNm which corresponds to a bending strain in the soffit of the deck of 425 με, assuming that the slab behaves as an elastic uncracked section. The ultimate traffic load effect for the concrete slab, based on the health monitoring data, is 310 με, and the resulting Fitness for Purpose Evaluation is 135%, or 1.35 in terms of the DCF.

5.4 Summary

The Fitness for Purpose Evaluation calculated for the superstructure of the Waipawa Bridge was 180%. The analytical rating evaluation of the structure (0.85 HO + 0.85 HN) is 140% and the analytical posting evaluation (0.85 HN) is 170%, as calculated by Infratech. Comparison with the posting is the most relevant as the rating is based on a heavier vehicle that is not representative of the normal heavy vehicle traffic using the bridge. The Fitness for Purpose Evaluation for the deck was 1.35. This is less than the empirical deck capacity of 1.6 which has been based on the assumption that the deck is in good condition, but it is higher than the rating calculated by the plate bending method. These ratings indicate that the bridge is fit for the heavy vehicle traffic using the bridge, and has considerable strength reserves.

The Fitness for Purpose Evaluation compares well with the posting evaluation calculated by Infratech. The reasons for this include:

- The heaviest vehicles on this route recorded during the monitoring period are typically inducing similar bending moments around 95% to 100% of the (0.85 HN) rating load. Overloading is well controlled on this route with the heaviest vehicles inducing similar bending strains to the known vehicle.
- The structure is behaving similarly to the model used in the analysis with similar distributions of load between the analytical method and the recorded data.

5.5 Comments

The evaluations for this bridge have assumed that composite action occurs between the slab and the steel girder. The analytical assessment of this bridge also identified that the shear connection was not sufficient to provide full composite action in the girders at the ultimate limit state. Comparing the strains recorded in the bottom flange with the analytical strains indicates that the girders are behaving as composite girders.

Future Health Monitoring programmes on structures of this type should include a strain gauge on the top steel girder flange to determine the location of the neutral axis of the section to confirm the degree of composite action. A displacement transducer to measure slip between the steel girder and the concrete slab would also be useful for the same purpose.

6. Conclusions

This report presents the details and results of the Health Monitoring programme applied to the Waipawa Bridge. A Fitness for Purpose Evaluation has also been derived for the bridge, based on the health monitoring data.

The theoretical analysis of the bridge found that midspan bending of the main girders and the performance of the deck were the critical issues associated with the performance of the bridge, and the Health Monitoring programme focused on the performance of these components.

Theoretical Analysis

The theoretical analysis of the structure found that the rating evaluation (0.85 HO + 0.85 HN) was 140% and the posting evaluation (0.85 HN) was 170%. These evaluations were significantly higher than the ratings given in the 1999 TNZ Structural Inventory. This may be related to assumptions made regarding the degree of composite action and inclusion of the kerbs in the strength calculations. The rating of the deck (0.74), based on plate bending, was similar to the rating from the Inventory, although the rating from the empirical method was higher. Based on this analysis, the Health Monitoring concentrated on the performance of the girders and the deck slab.

Health Monitoring Results

The Health Monitoring investigation of the Waipawa Bridge found that:

- Although the spans were assumed to behave as simply supported spans, some continuity exists between the span that was monitored and the adjacent span.
- The heaviest vehicles are typically inducing effects around 95% to 100% of the 0.85 HN loading, and a significant portion of the vehicles on this route are lighter than these vehicles. Overloading is well controlled on this route.
- Some differences between the analytical distribution of load and the recorded distributions of load could be related to differences in vehicle position in the lane adjacent to the footpath.
- The impact factor of 1.25 obtained from the Bridge Manual is appropriate for this bridge. The significant differences in the values obtained for vehicles travelling in different directions may be related to the effect of the road profile.

Fitness for Purpose Evaluation

- The Fitness for Purpose Evaluation for this bridge, based on the critical midspan bending, was 180%. This indicates that the bridge is safely carrying the heavy vehicle traffic using the route, with significant strength reserves.
- The transverse cracking in the deck slab is not significantly affecting the performance of the slab and the Fitness for Purpose Evaluation of the deck slab was 135%, or 1.35 in terms of Deck Capacity Factor.

7. Recommendation

The cracking of the deck should be addressed only from a durability perspective to minimise future bridge maintenance costs and to prolong the life of the bridge.

8. References

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