Health Monitoring of Superstructures of New Zealand Road Bridges:

Rakaia Bridge, Canterbury

Transfund New Zealand Research Report No.167

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Rakaia Bridge, Canterbury

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Transfund New Zealand Research Report No. 167

ISBN 0-478-11575-X
ISSN 1174-0574

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Andersen, R.P., Roberts, W.S., Heywood, R.J., Heldt, T.J. 2000. Health monitoring of superstructures of New Zealand road bridges: Rakaia Bridge, Canterbury. *Transfund New Zealand Research Report No. 167*. 48pp.

Keywords: bridges, bridge dynamics, bridge health, bridge loads, heavy vehicles, loading, monitoring, New Zealand, performance, roads, superstructure, traffic

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Acknowledgments

This project has been greatly assisted by the support and co-operation of many people and organisations. In particular, Infratech Systems & Services Pty Ltd, gratefully acknowledge the technical reviewers, Mr Frank McGuire (Transit New Zealand) and Dr John Fenwick (Department of Main Roads, Queensland), for their valuable insight and assistance with the development of this report. The support and assistance of many people were required to complete the field work, in particular Mr Derek Dumbar (TD Haulage), Mr Colin Stewart (Transit New Zealand), Mr John Reynolds (Opus International Consultants), and to Freightways Express Ltd for use of their heavy vehicle. The assistance and support of the staff of Deloitte Touche Tohmatsu is also appreciated.

Infratech Systems & Services acknowledges the support of Transfund New Zealand, and of their staff involved in the project.

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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 Rakaia Bridge, on State Highway 1S, crosses the Rakaia River about 50 km south of Christchurch, Canterbury Region, South Island. It was selected because it is an aging (built in 1939), two-lane, concrete-girder bridge with integral guardrails, and has a low strength evaluation. It is typical of a significant proportion of New Zealand's bridge infrastructure. Also, because it is the main crossing of this wide braided river, and is very long (1757 m), this particular bridge represents a major asset in terms of New Zealand's transport system.

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.

Theoretical Analysis

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 its performance. The assessment of the superstructure found that the 0.85 HO* rating evaluation is 67%, the 0.85 HN* posting evaluation is 84%, and that the Deck Capacity Factor (DCF) is 0.97. These theoretical evaluations did not include any strength contribution from the guardrails.

Health Monitoring Results

The findings from the Health Monitoring programme show that:

- The guardrails are contributing to the strength of the bridge and have been included in the Health Monitoring evaluation of this bridge.
- The ambient heavy vehicle traffic is inducing bending moments in the bridge that are 5% to 10% higher than the 0.85 HN vehicle, indicating that some of the traffic on this route is heavily loaded.
- The recorded strains in the girders are approximately 50% to 60% of the theoretically predicted strains. This is related to a number of effects including the contribution of the guardrails to bridge strength, and possibly to some bearing-restraint continuity effects. The guardrails were not included in the theoretical analysis.

* HO Highway overweight vehicles; HN Highway normal vehicles

- The uneven road profile on the northern approach to the bridge is increasing the dynamic effects on the monitored span. The dynamic increment for southbound traffic is significantly higher than that for northbound traffic.

  The highest measured impact factor was 28% for this bridge, which is similar to the impact factor of 1.3, used to determine the load rating, and detailed in the Transit New Zealand 1994 Bridge Manual. Improvement of the road profile on this bridge may reduce the effects of heavy vehicles on it.

Fitness for Purpose Evaluation

The Fitness for Purpose Evaluation for the Rakaia Bridge, based on the critical midspan bending of the main girders, was 137%.

The Fitness for Purpose Evaluation for the deck was 118%, or 1.18 in terms of the DCF.

This evaluation indicates that the bridge is safely carrying the heavy vehicle traffic currently using this route.

Recommendations

The recommendations obtained from this investigation are for:

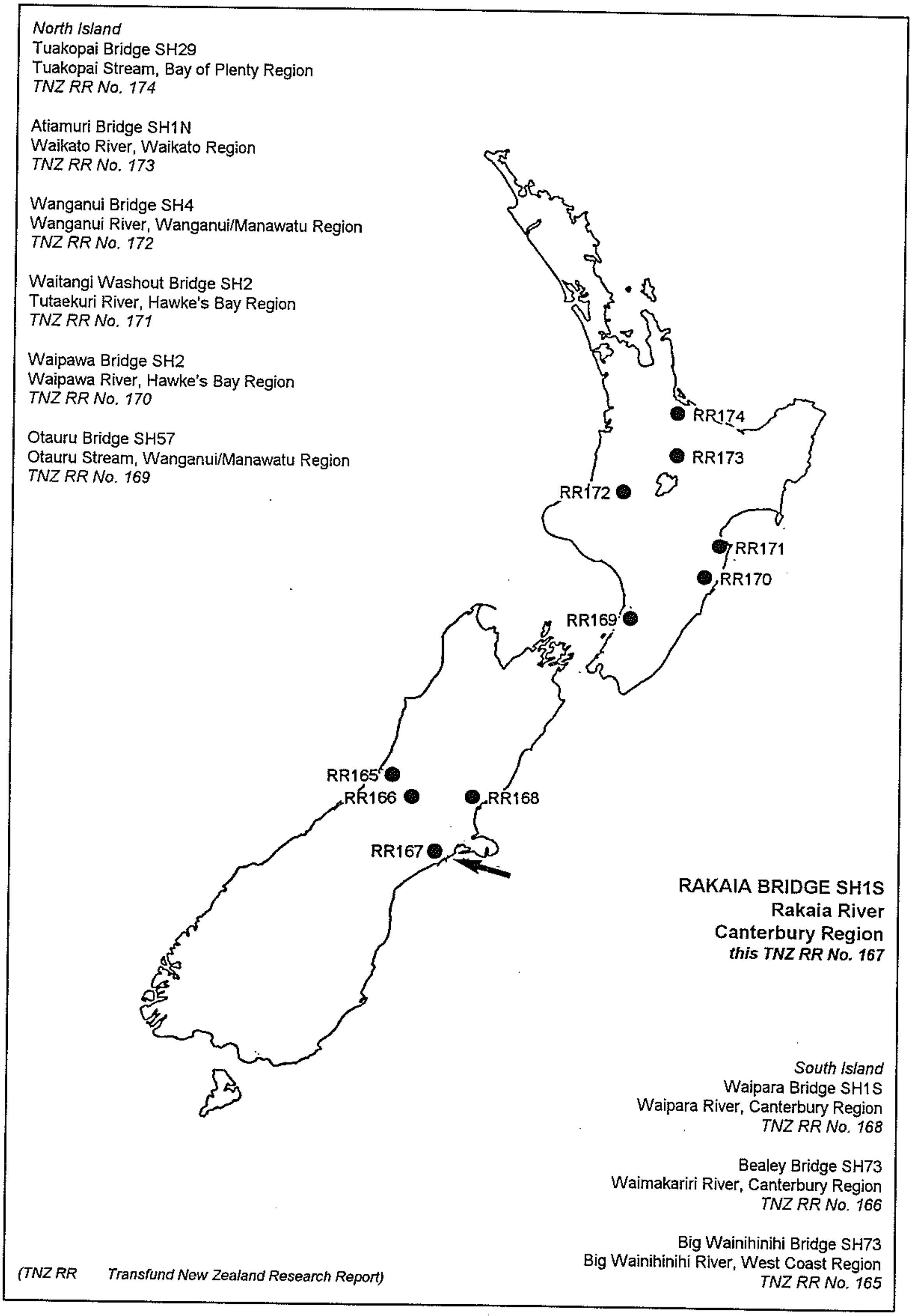
- Further investigation of the contribution of the guardrails to the strength of this bridge, and extrapolating the results to other bridges. Substantial economic benefit may result from including the contribution of guardrails to bridge capacity.
- Investigation of the performance of Girder 4 by increasing the monitoring period (to possibly one month).
- Consideration of whether the Rakaia Bridge is to be posted with a load limit. Although the theoretical rating suggests that this bridge should be posted, the Health Monitoring indicates that it is not required.

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 Rakaia Bridge, on State Highway 1S, crosses the Rakaia River about 50 km south of Christchurch, Canterbury Region, South Island. It was selected because it is an aging (built in 1939), two-lane, concrete-girder bridge with integral guardrails, and has a low strength evaluation. It is typical of a significant proportion of New Zealand's bridge infrastructure. Also, because it is the main crossing of this wide braided river and is very long (1757 m), this particular bridge represents a major asset to New Zealand's transport system. The Fitness for Purpose Evaluation indicates that the bridge is safely carrying the heavy vehicle traffic currently using this route.

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



1. Introduction

1.1 Bridge Health Monitoring

Bridge Health Monitoring is a method of evaluating the ability of a bridge to perform its required task, or 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 merging 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.

The 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 Rakaia Bridge on State Highway (SH) 1S, which crosses the Rakaia River about 50 km south of Christchurch, Canterbury Region, South Island of New Zealand (Figure 1.1). The reasons for choosing this bridge for the representative sample were:

- It is an aging (built in 1939), two-lane, reinforced-concrete structure.
- It has a low conventional strength evaluation.
- Because it is the main crossing of this wide braided river, and is very long (1757 m), this particular bridge represents a major asset in terms of New Zealand's transport system.
- It is also typical of a significant proportion of New Zealand's bridge infrastructure.

The objective of this investigation was to evaluate the Fitness for Purpose of the Rakaia 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 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 is based principally on the following components of the superstructure:

- Midspan bending strength of the main concrete girders.
- Shear strength of the main concrete girders.
- Capacity of the concrete deck.

The substructure was not evaluated in 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.

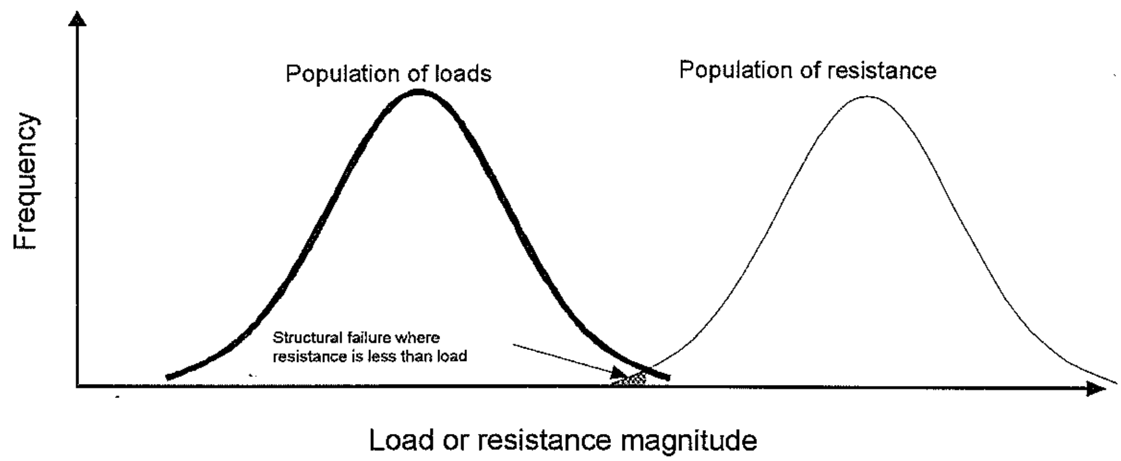


Figure 2.1 Statistical representation of structural failure.

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** based 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 of a bridge and decks 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
ϕ = Strength Reduction Factor
R_I = Section Strength
γ_D = Dead Load Factor
DL = Dead Load Effect
γ = Load factors on other effects
γ_o = Overload 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 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 the standard equations to be combined 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 Rakaia 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 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 Rakaia Bridge on SH 1S consists of 144 simply supported spans, each with four reinforced-concrete girders, supporting a reinforced-concrete deck. The concrete deck has integral kerbs and guardrails. The typical span length is approximately 12.2 m with two of the spans being 12.5 m long. Construction of the 1757 m-long bridge was completed in 1939. The bridge is illustrated in Figure 3.1, and is the longest road bridge in New Zealand.

Figure 3.1 Rakaia Bridge, Canterbury, South Island, New Zealand.

The road profile leading up to the northern approach to the bridge is deteriorating, and this approach is illustrated in Figure 3.2. The bump in the road profile is causing the vehicles to bounce onto the first span which increases the dynamic loads being applied to Span 1. For this reason this span of the bridge was monitored though the road profile for the remainder of the bridge is also of concern.

Figure 3.2 Road profile at the northern approach to Span 1 of the bridge.

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

- Bridge (superstructure) Class 76%
- Deck Capacity Factor (DCF) 0.93

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, an analysis of the structure was conducted using the 0.85 HN and 0.85 HO rating and posting loads (see section 2.1 of this report), as specified in the Bridge Manual. Results from an analysis using the "known vehicle" (section 2.4.2) used in the Health Monitoring programme are also included. Details of this known vehicle are given in section 4.2 of this report.

A typical span of the bridge superstructure was investigated using a "grillage analysis"[1]. The grillage analysis assumed that the girders are simply supported. The dimensions of the structure used in the analysis were taken from the "as constructed" plans, which were confirmed by on-site measurements.

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

The material properties for the concrete were not available. The properties used for the concrete were obtained from Section 6.3.4 of the Bridge Manual, and the material properties (nomenclature as in the Bridge Manual) used in the analysis of this bridge are as follows:

- Concrete Girders and Deck f'_c = 17 MPa, E = 20 800 MPa
- Steel Reinforcement f_y = 250 MPa, E = 200 000 MPa

The grillage model did not include the stiffness of the guardrails with the edge girders. However the stiffness of the kerbs was included.

3.2.1 Girder Bending

The maximum bending moment in the critical outer girders resulting from the dead load is 366 kNm/girder. A summary of the maximum bending moments in the critical edge girder resulting from the various loads applied to the grillage model is presented in Table 3.1. The results in the table are not factored, and they represent the maximum bending moment in a single edge girder with the vehicle at the greatest allowable eccentricity.

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

Load	Bending Moment (kNm)
Dead Load	366
Known Vehicle	139
2x 0.85HN Vehicles (Posting Load)	264
0.85HO + 0.85HN Vehicles (Rating Load)	422

The bending capacity of the concrete girders of the superstructure, calculated in accordance with Section 8 of the Concrete Structures Standard (NZS 3101: Part 1 1995), is 1021 kNm. This strength does not include any contribution from the kerbs or the guardrails.

3.2.2 Girder Shear

The shear force in each girder was found using the grillage analysis, and the results are presented in Table 3.2. The shear capacity (ϕV_n) of the main girders, calculated in accordance with Section 9 of the Concrete Structures Standard (NZS 3101: Part 1 1995), is 545 kN.

Table 3.2 Grillage analysis results for shear (kN) in the girders.

Load	Shear Force (kN)
Dead Load	123
Test Vehicle	64
2x 0.85HN Vehicles (Posting Load)	111
0.85HO + 0.85HN Vehicles (Rating Load)	176

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 of the Bridge Manual).

Analysis found that the HO wheel load was critical with the resulting bending moment being 28 kNm, assuming the deck is continuous over the girders. The effective width of deck slab resisting this footprint was assumed to be 1.4 m, and the bending capacity of the deck at the ultimate limit state (ϕM) was 41 kNm.

3.2.3.2 *Shear*

The shear strength of the deck slab was calculated using Section 9 of the Concrete Structures Standard (NZS 3101: Part 1 1995). The shear capacity of the deck was found to be 131 kN. The maximum shear force (V*) applied to the deck by the HO wheel loading is 70 kN.

3.2.3.3 *Empirical Method*

The capacity of the deck can also be calculated using empirical methods that are presented in the Bridge Manual (Section 6.5.2), and determined from Figures 6.1 to 6.5 in that section. In the case of the Rakaia Bridge, the empirical method is not applicable because the minimum strength of the concrete must be at least 20 MPa. However, the strength of 17 MPa was based on the recommendation of the Bridge Manual but the actual in-situ strength of the deck is expected to be higher than 17 MPa, and probably at least equal to the 20 MPa minimum required for the empirical method..

The Deck Capacity Factor, based on the empirical capacity of the deck and on a 20 MPa concrete strength, is 1.6.

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 2.3 of this report, and the results are presented in Tables 3.3 and 3.4. The evaluation has been assessed for the bending and shear in the girders and deck. The tables also present a comparison of the load rating calculated by Infratech Systems & Services (Infratech), and the load rating recorded in the current (1999) TNZ Structural Inventory. A value of 1.3 was used for the impact factor in calculating the load ratings. Impact factors are not included in the rating loads presented in the tables, but they are included in the rating and posting evaluation calculations.

Table 3.3 Summary of theoretical load evaluations for the critical edge girders.

Mode of Failure	ϕ Ultimate Capacity	0.85 HO Rating Load	0.85 HN Posting Load	Dead Load	0.85 HO Rating (Infratech)	0.85 HN Posting (Infratech)	Rating (Structural Inventory)
Girder Bending	1021kNm	422kNm	264kNm	366kNm	67%	84%	76%
Girder Shear	545kN	176kN	111kN	123kN	112%	140%	

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

Mode of Failure	ϕUltimate Capacity	0.85 HO Rating Load	0.85 HO Rating (Infratech)	Rating (Structural Inventory)
Deck Bending	41kNm	28kNm	0.97%	0.93%
Deck Shear	131kN	70kN	0.98%	

The overall rating of the girders is taken as the minimum rating of all the components. For this bridge, the rating is the minimum of the ratings based on shear and bending (67%), and the critical failure mode is midspan bending of the girders. This compares to the rating of 76% which is documented in the TNZ Structural Inventory.

The Deck Capacity Factor calculated by Infratech was 0.97 which is similar to the value of 0.93 quoted in the TNZ Structural Inventory. A value for the DCF of 1.6 was also calculated assuming a concrete strength of 20 MPa.

The posting evaluation of this structure is less than 100%, and therefore the normal practice would be to post the bridge. It is understood that the bridge is currently not posted.

3.4 Summary

The Rakaia Bridge, in Canterbury, 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 determines the rating of the superstructure.

The deck capacity is also governed by bending effects, although the capacity, using the empirical method but based on an assumed concrete strength of 20 MPa, gives a higher result.

Based on the results from this analysis, the Health Monitoring programme concentrated on evaluating the Fitness for Purpose for the girders based on midspan bending. The evaluation of the deck was also based on bending effects.

4. Health Monitoring Programme

The programme of Health Monitoring on the Rakaia Bridge involved two components:

- Short-term health monitoring of the ambient heavy vehicle traffic for a period of approximately 2 days.
- 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 Rakaia Bridge.

4.1 Instrumentation

The instrumentation installed on the bridge included seven Demountable Strain Gauge transducers and one Deflection transducer. The instrumentation was installed on the extreme northern span adjacent to the abutment, and the locations of this instrumentation are illustrated in Figure 4.1.

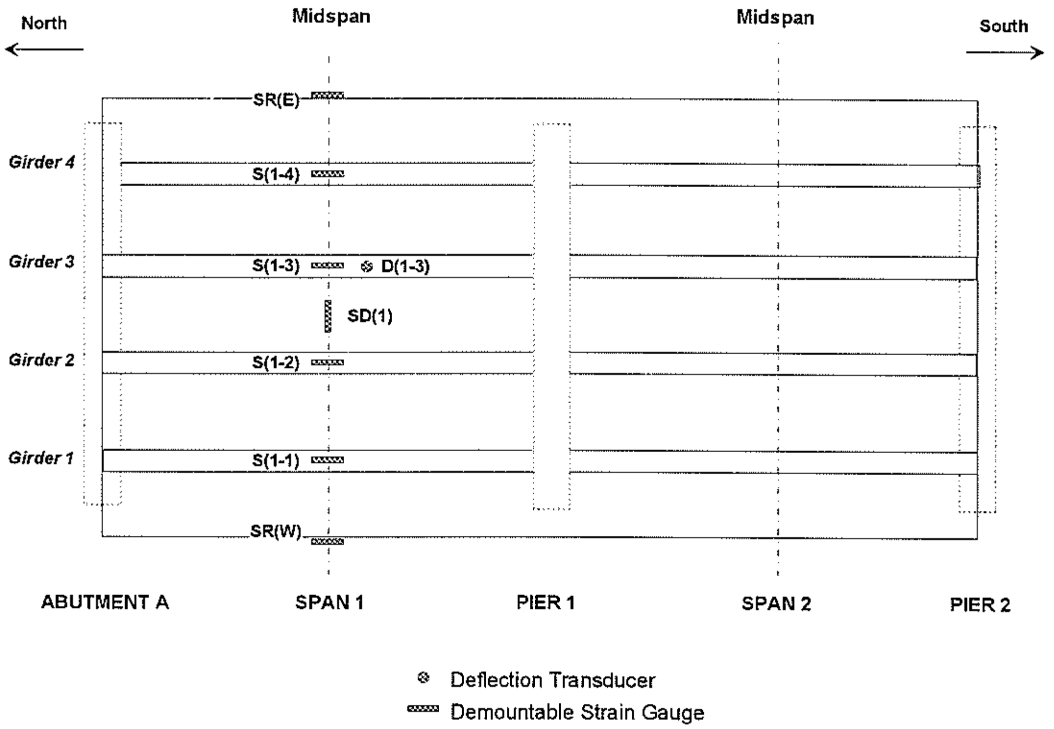


Figure 4.1 Instrumentation plan for the Rakaia Bridge.

S - strain transducers on girders; SR(E), (W) - strain transducers on guardrails (east, west);
D - deflection transducer; SD - transverse strain transducers

Figure 4.2 shows a Demountable Strain Gauge transducer and a Deflection transducer installed on Girder 3. The deflection transducer was installed on Girder 3 to provide additional information on the performance of this girder.

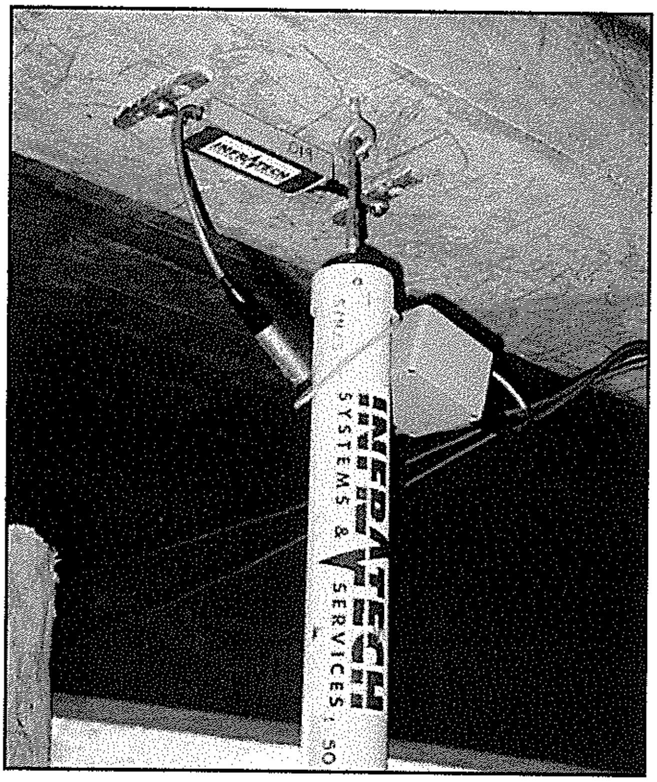


Figure 4.2 Instrumentation on Girder 3.

Two demountable strain gauge transducers, SR(E) and SR(W), were installed on the guardrails, to determine if the guardrails are contributing to the strength of the structure. The detail of the positions of this instrumentation is illustrated in Figure 4.3, and Figure 4.4. shows one of the transducers on the guardrail.

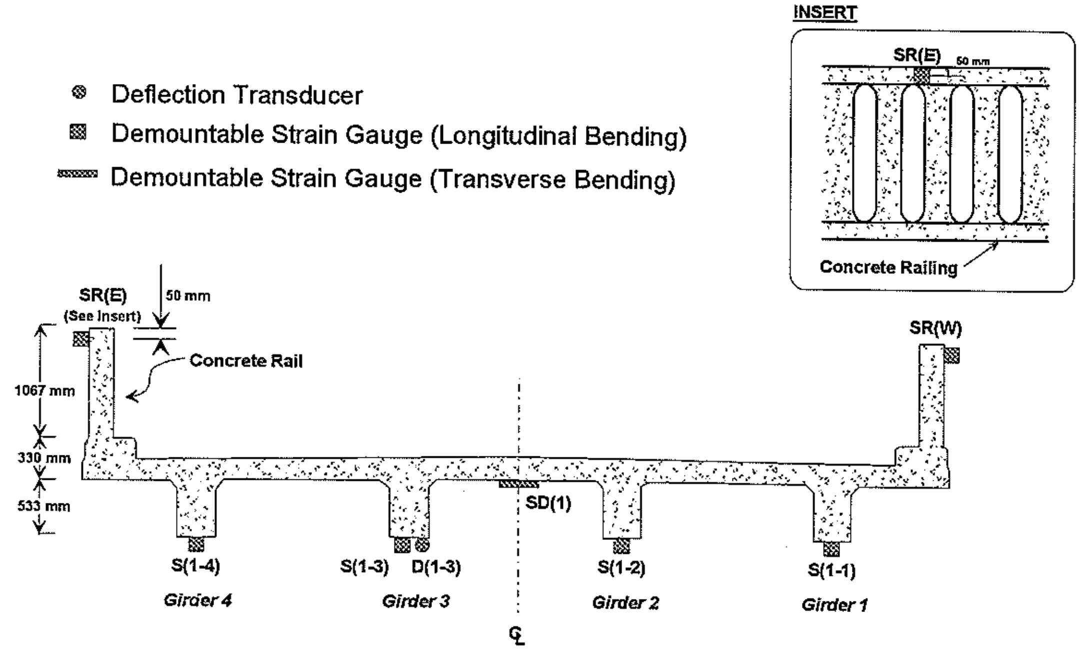


Figure 4.3 Cross section of the bridge and the instrumentation.

Transducer SD(1) was installed to record the transverse bending strains in the deck, and was positioned between Girders 2 and 3, which corresponds to the centre of the marked roadway. This made it possible to record the deck response to the heavy vehicles travelling in either direction (north or south).

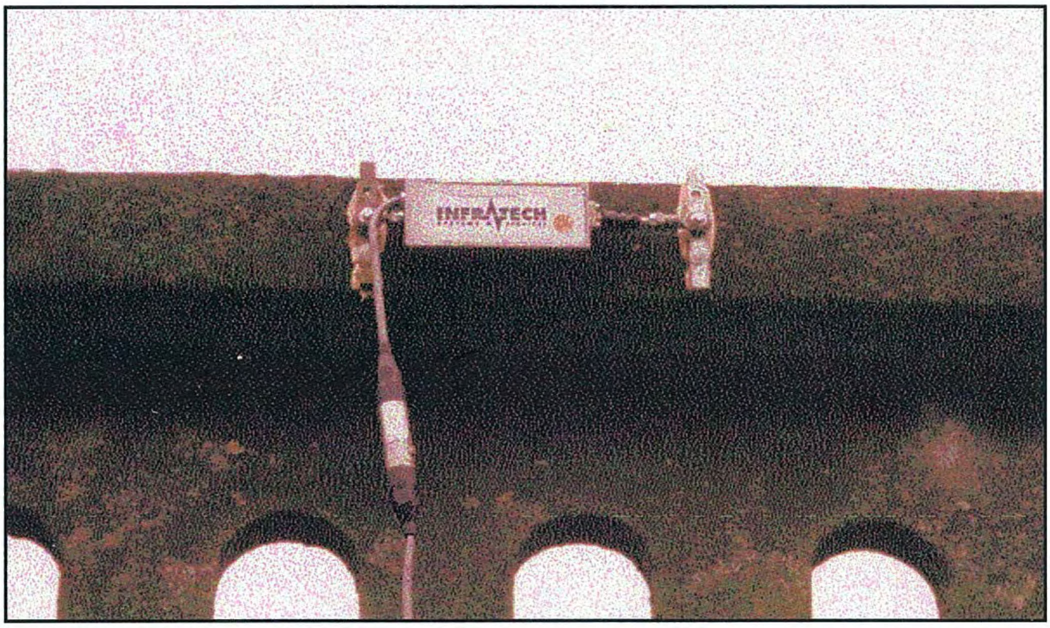


Figure 4.4 A typical transducer installed on the guardrail.

The demountable strain gauge transducers (gauge length 230 mm) used on the girders and deck measured strain at a point 20 mm below the soffit. The results have been corrected to represent the strain in the soffit of the girders and deck. The sign conventions used throughout this report include positive values for tension strains and negative values for compressive strains. For the deflection transducer, positive deflections indicate a downwards deflection of the structure.

4.2 Procedure

The health monitoring of the structure began on Monday 23 November 1998 and continued until Wednesday 25 November 1998, giving a total monitoring period of approximately 46 hours. During the 2-day monitoring period, the response of the bridge to 1032 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 Wednesday 25 November 1998. The vehicle (Figure 4.5) was supplied by Freightways Express, and the axle weights and configuration are illustrated in Figure 4.6.

Figure 4.5 The known vehicle used for behavioural testing.

Figure 4.6 Axle mass and configuration of the known vehicle.

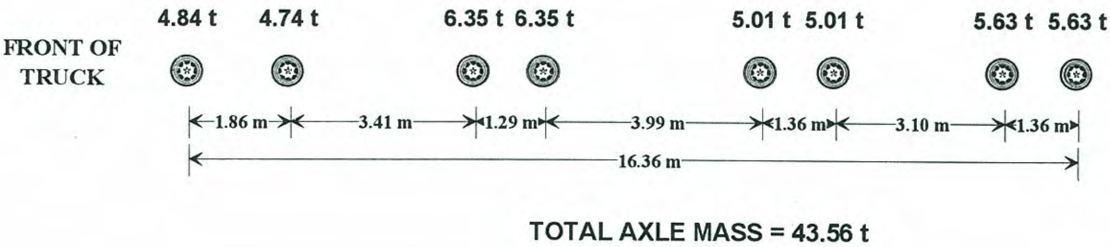


Figure 4.7 Known vehicle in the normal traffic lane during the testing.

The testing with the known vehicle was conducted by recording the response of the bridge to the vehicle as it passed over the bridge at various speeds. The tests were conducted with the vehicle travelling in both directions (east, then west), at a crawl (10 km/h), 20 km/h, 40 km/h, 60 km/h, 80 km/h, and 100 km/h.

The lateral position of the known vehicle was in the normal lane as shown in Figure 4.7. Testing was completed by slowing the traffic in each direction or in some cases stopping it for a few minutes at a time. This ensured minimal traffic interruptions and also allowed the 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) for the midspan bending strains recorded during the health monitoring for the passage of the heaviest vehicle. The response is presented in Figure 4.8. The waveforms show some dynamic response after the vehicle had passed over the instrumented span, and significant dynamic response is also evident while the vehicle was on the instrumented span.

The scatter diagram for midspan bending strains (Figure 4.9) represents the maximum strains recorded during the passage of each heavy vehicle for the entire Health Monitoring period. These plots give an indication of the characteristics of the heavy vehicles travelling over the bridge including distribution of mass and the number of heavy vehicles travelling this route. The gap in the data during in the afternoon of 24 November is from monitor downtime, not to an absence of traffic.

The scatter diagram presented in Figure 4.9 displays consistently higher responses from transducer S(1-4) in comparison to S(1-1) located on the opposite side of the bridge. This may be related to several issues and is discussed further in section 4.3.2 in this report.

A waveform for the deflection response of Girder 3 for the heaviest vehicle recorded is illustrated in Figure 4.10, and the scatter diagram for this transducer is presented in Figure 4.11.

Figure 4.10 Waveform for deflection transducer D(1-3) for event recorded at 6.49am, 25 November 1998.

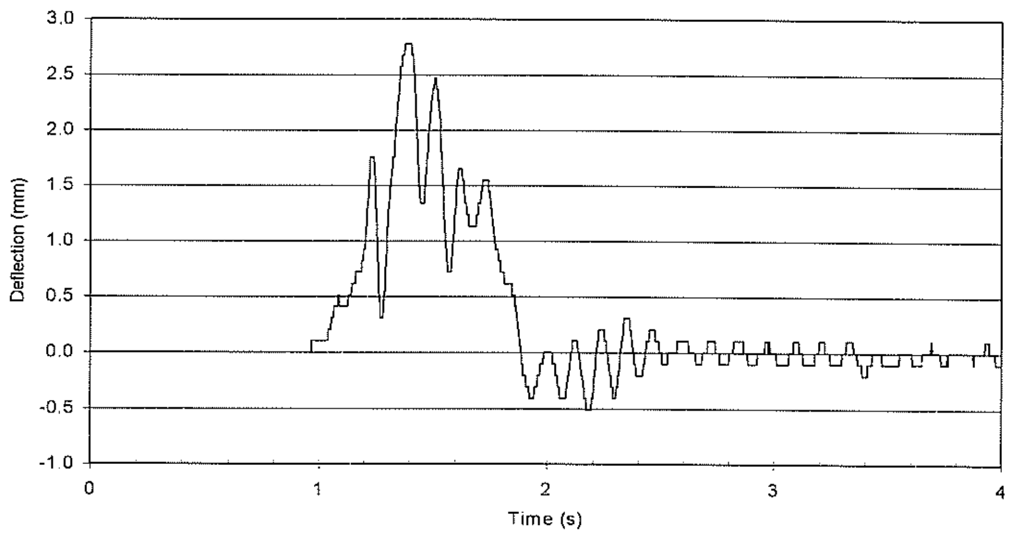
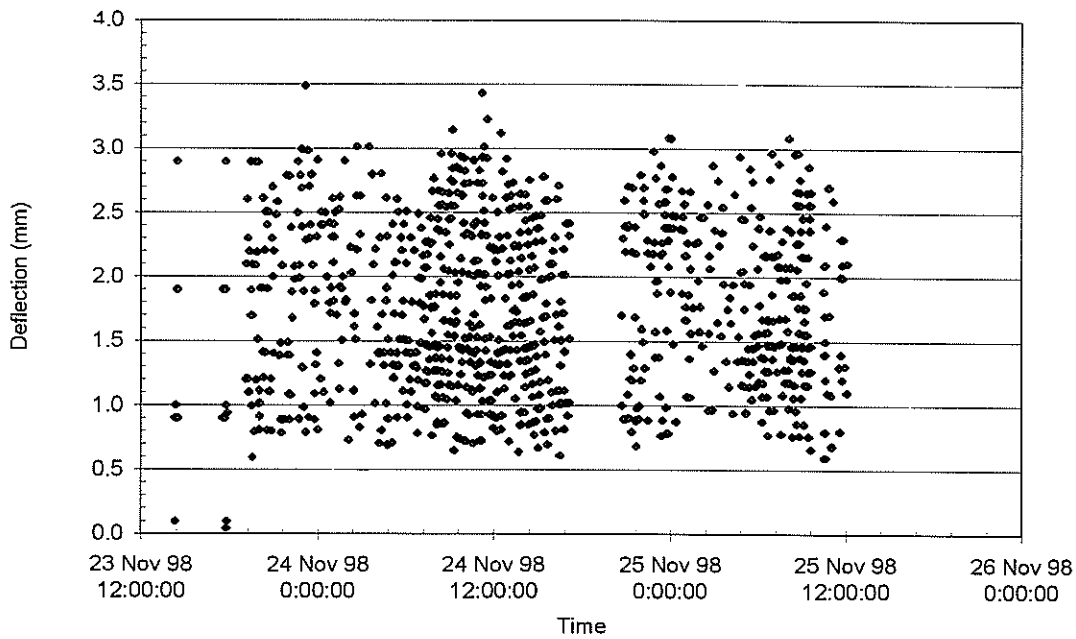


Figure 4.11 Scatter diagram for deflection transducer D(1-3).



4.3.2 Guardrail Response

The waveforms for the transducers installed on the guardrails are illustrated in Figure 4.12. The plot shows the response of the structure as the vehicle travelled south and shows that the guardrails experienced compression responses as expected. A scatter diagram for these transducers is presented in Figure 4.13.

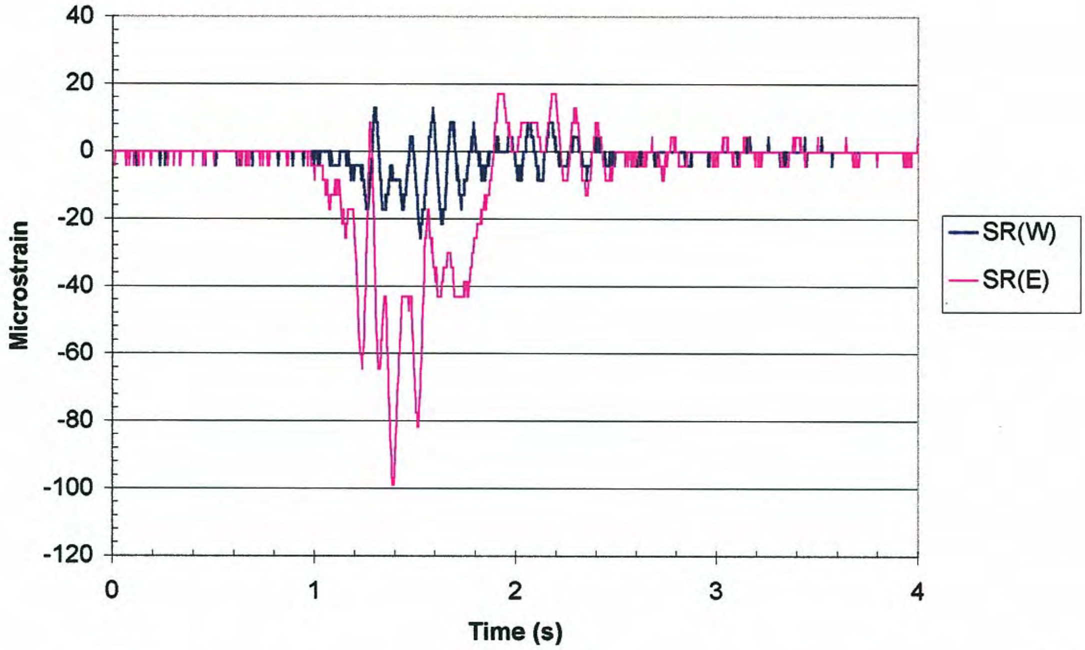


Figure 4.12 Strain response versus time for transducers installed on the guardrails, for event recorded at 6.49am, 25 November 1998.

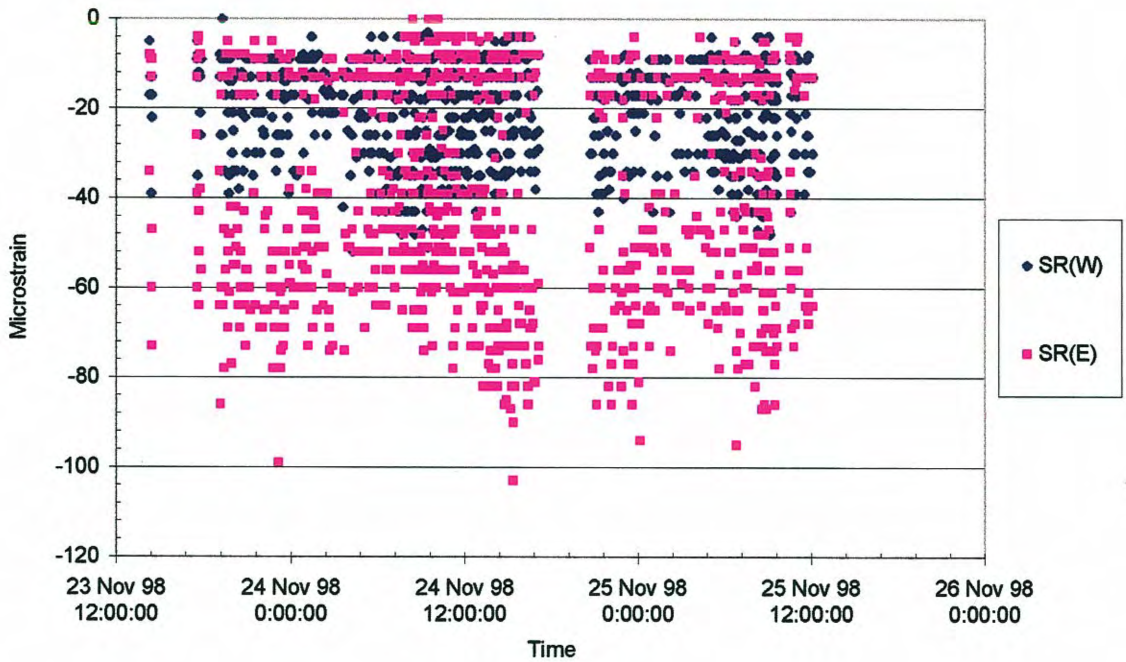


Figure 4.13 Scatter diagram for transducers installed on the guardrails.

These results indicate that the guardrails are contributing to the strength of the structure in terms of midspan bending. For each event, the magnitude of the strain in the guardrails is approximately half of the maximum strain in the adjacent outside girders of the structure.

4.3.3 Deck Response

A typical waveform for the deck transducer is illustrated in Figure 4.14. It shows the response of the deck as the wheel passed over the transducer. The waveform also illustrates the presence of some dynamic action as the wheels passed over the transducer location.

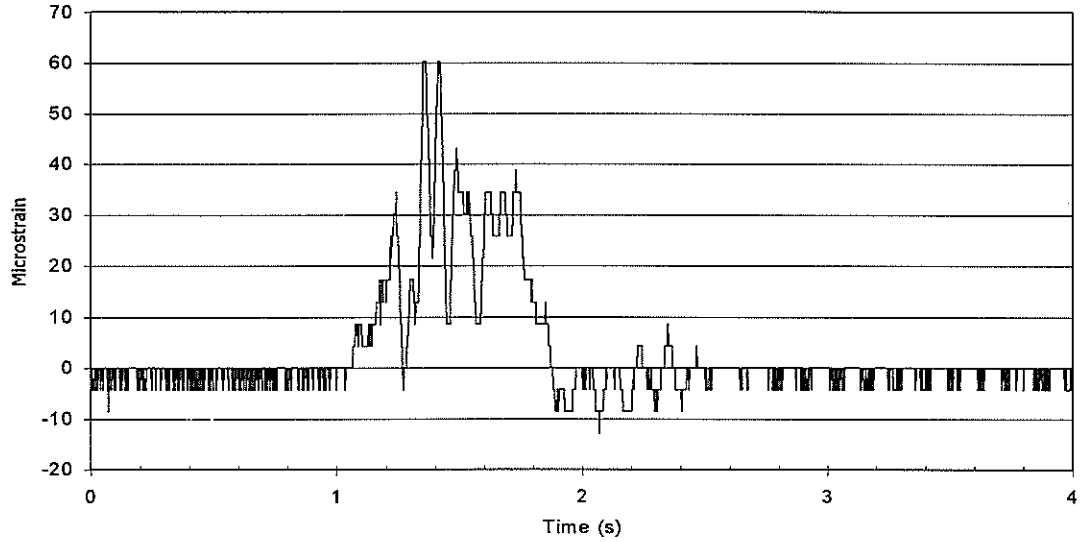


Figure 4.14 Waveform for deck transducer SD(1) for event recorded at 6.49am, 25 November 1998.

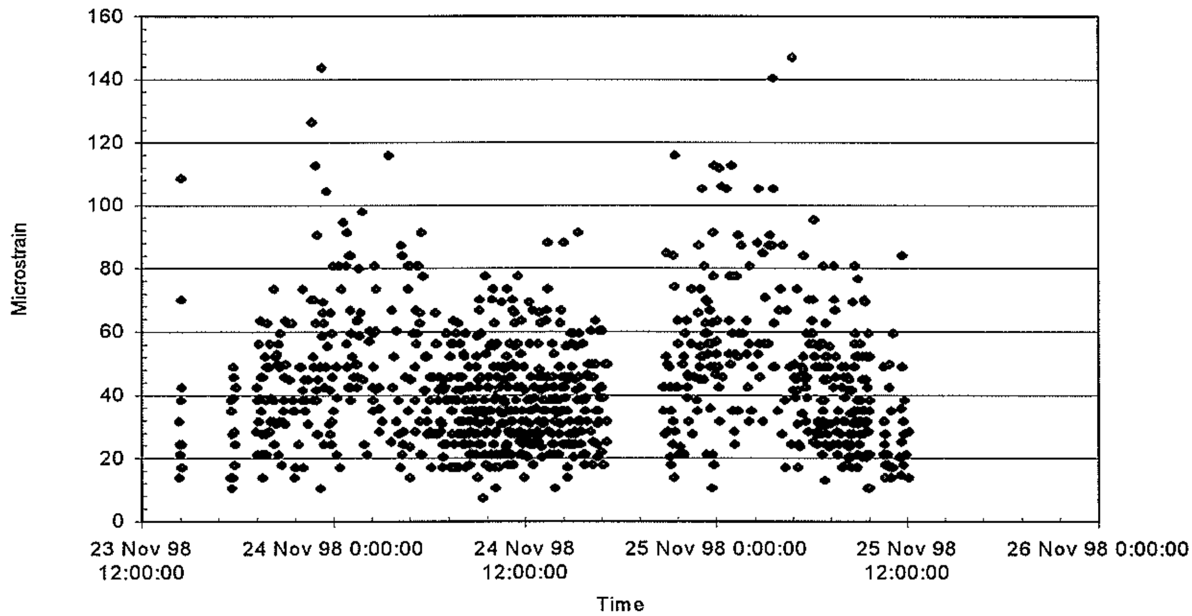


Figure 4.15 Scatter diagram for deck transducer SD(1).

The scatter diagram for the deck transducer is presented in Figure 4.15, and it shows a varied response for each event. This is characteristic of most deck responses and is due to the sensitivity of the response to the wheel position, which must be directly between girders for maximum bending.

4.3.4 Extrapolated Data

The data from the scatter diagrams can also be plotted on a histogram that incorporates a cumulative distribution. An example, for transducer S(1-4), is presented in Figure 4.16. The histogram illustrates two separate sections or populations of data, which is characteristic of traffic travelling in opposite directions on different sides of the bridge. By separating the data into directions, the data relevant to each transducer can be plotted and a more accurate ultimate load effect can be determined for each girder.

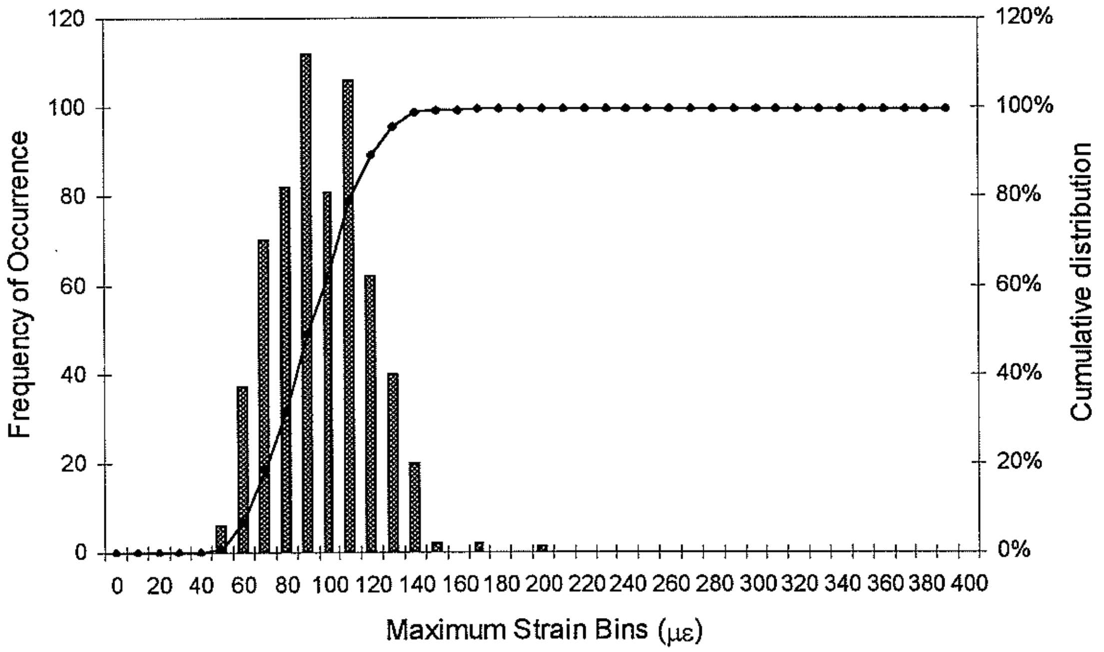


Figure 4.16 Histogram and cumulative distribution function for midspan transducer S(1-4).

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 transducers measuring midspan bending strain is presented in Figure 4.17. This figure presents the data separated into opposite directions: north for transducers S(1-1) and S(1-2), and south for transducers S(1-3) and S(1-4). 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 a data plot crosses the horizontal axis represents the average (mean) strain. 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, that has been outlined in section 2.4 of this report.

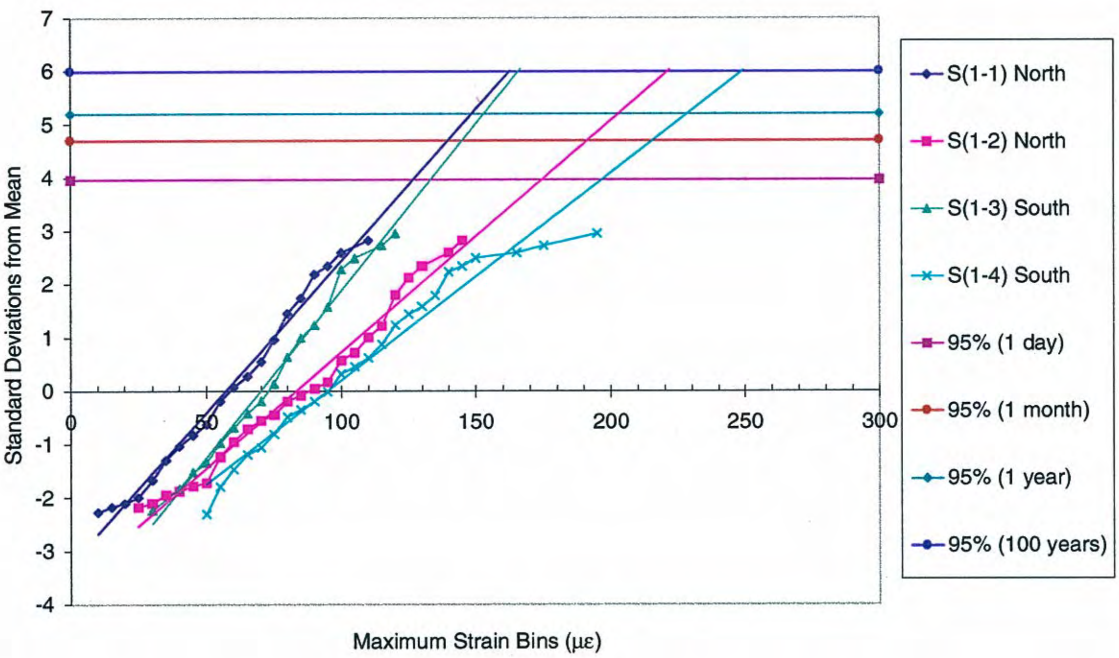


Figure 4.17 Inverse normal plot for strain transducers installed at the midspan.

The inverse normal plot shows that the strain extrapolated for the 95% confidence limit for 100 years (ultimate traffic load effect) is the greatest for the midspan transducer S(1-4). The extrapolated value is approximately 250 με.

Higher strains were also recorded in Girders 3 and 4 compared with Girders 1 and 2. This may indicate that the traffic is more heavily loaded travelling south or that there are differences in the structural behaviour.

Figure 4.18 Inverse normal plot for transducers installed on the guardrails.

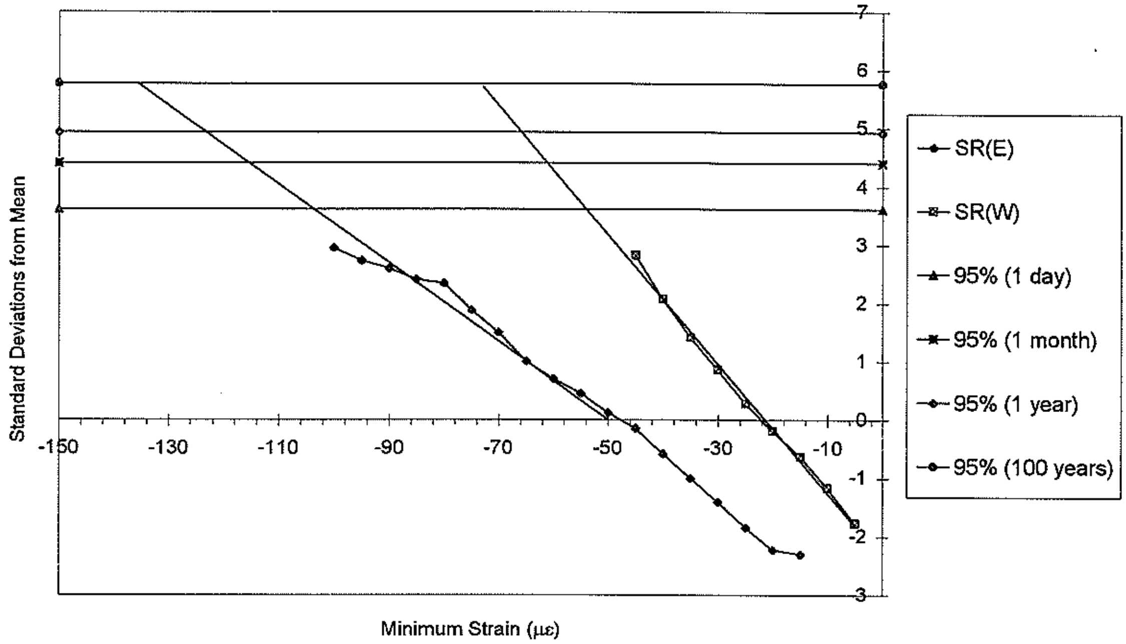
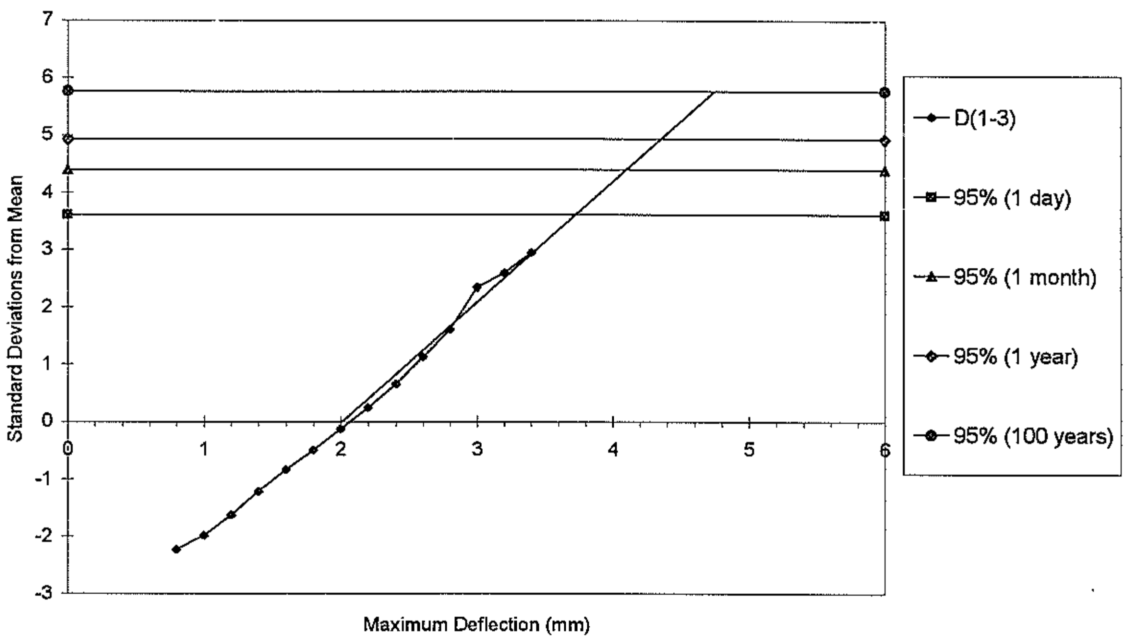


Figure 4.19 Inverse normal plot for deflection transducer D(1-3).



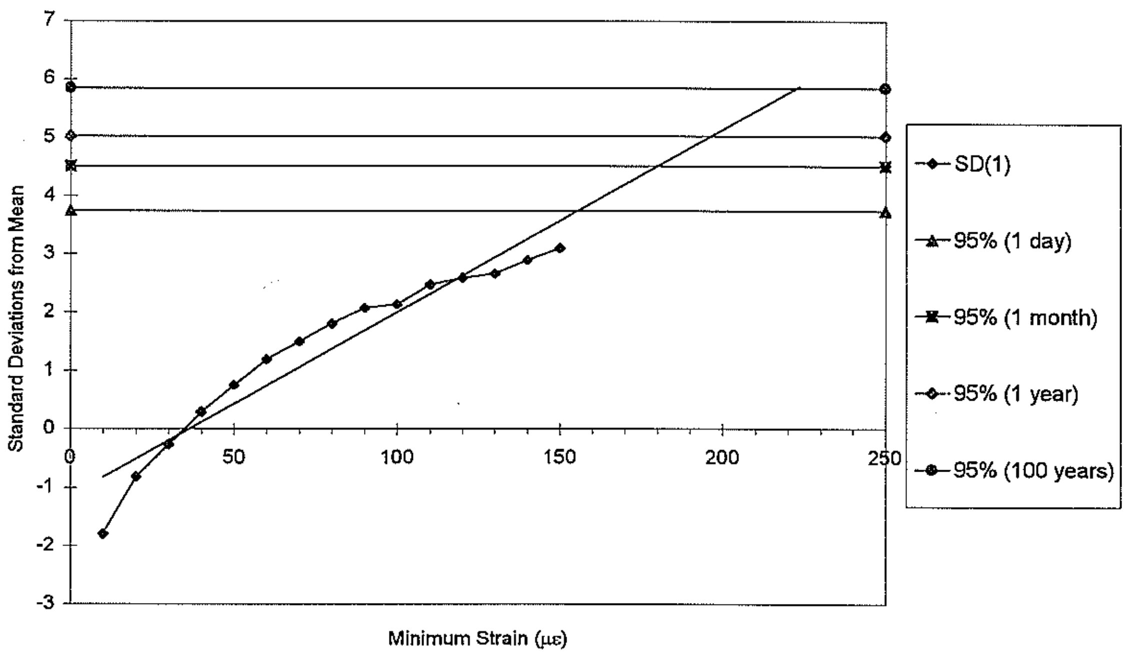
A distinct change is seen in the behaviour of the response from transducer S(1-4) at around 150 $\mu\varepsilon$. This is also evident in the other transducers although it is not as dramatic. This change may represent the effect of a different population of vehicles (overloaded vehicles for example), a change in the structural behaviour (uncracked state to cracked state), or a change in behaviour associated with the guardrail. With the current data, this change in behaviour was represented by only three vehicles. A longer monitoring period is recommended to confirm the reason for this change in behaviour.

The inverse normal plots for the transducers installed on the guardrails are illustrated in Figure 4.18. These data show that significantly higher strains were recorded in the eastern guardrail on the side of the bridge corresponding to Girders 3 and 4.

The inverse normal plot for the deflection transducer installed on Girder 3 is presented in Figure 4.19. The results from this transducer show similar characteristics indicating that the strain and deflection behaviour is consistent. The recorded deflections are well below typical limits for service load deflections of Span/800 (AUSTROADS 1992) which, for this bridge, corresponds to a deflection of 15 mm.

The inverse normal plot for the transducer measuring transverse bending strain in the deck is presented in Figure 4.20. The shape of the curve is less linear than the plots for the other girder transducers. This may be related to the variation in responses for each event, because the deck response is sensitive to the wheel position.

Figure 4.20 Inverse normal plot for bending strains in the deck.



The maximum results, along with the extrapolated results (95% confidence limit for 100 years) for all transducers, are presented in Table 4.1.

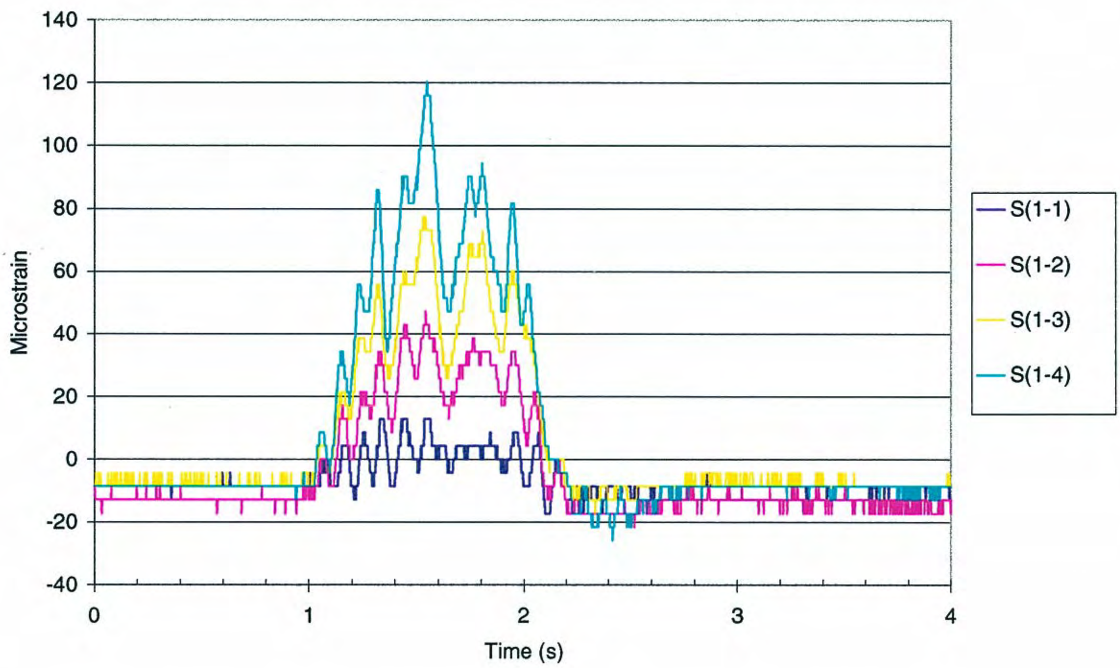
Table 4.1 Extrapolated data (strains με, deflections mm) obtained from inverse normal distribution.

Transducer	Maximum Recorded Value (Health Monitoring)	Extrapolated Value (95% confidence limit) for 100 years
	Strain (με)	
S(1-1)	109	160
S(1-2)	147	220
S(1-3)	118	170
S(1-4)	195	250
SR(E)	–100	–140
SR(W)	–45	–75
SD(1)	150	220
	Deflection (mm)	
D(1-3)	3.5	4.7

4.4 Known Vehicle Testing

A typical waveform for the passage of the known vehicle (travelling south) is presented in Figure 4.21. The dynamic response of the structure while the vehicle was on the instrumented span is again the dominating feature.

Figure 4.21 Typical waveform for the southbound passage of the known vehicle.



The known vehicle testing was performed at vehicle speeds ranging between a crawl and 100 km/h, and the maximum strain and deflection responses that each transducer recorded are presented in Table 4.2.

Table 4.2 Maximum recorded responses (strains (με), deflections (mm)) for known vehicle testing.

Transducer	Maximum Response
S(1-1)	86 με
S(1-2)	112 με
S(1-3)	86 με
S(1-4)	129 με
SR(E)	–43 με
SR(W)	–69 με
SD(1)	43 με
D(1-3)	2.4 mm

The distribution of strain into each of the girders from the known vehicle data is presented in Figure 4.22. The distribution presented is consistent with the data collected from health monitoring of the ambient heavy vehicle traffic. The distribution shows that higher strains were recorded in Girder 2 (middle girder) for vehicles travelling north, while for vehicles travelling south the maximum strain was recorded on Girder 4 (edge girder). The structure is symmetrical and the marked lanes also follow this pattern, with the vehicles typically travelling in the standard marked lanes. The differences in the strain distributions are expected to relate to differences in the structural behaviour of the girders.

Figure 4.22 also illustrates the results from the grillage analysis that included one vehicle of the same axle and load configuration as the known vehicle. This grillage analysis included the effects of the kerb but not of the guardrail, and also does not include dynamic effects. The vehicle position for the grillage was 600 mm out from the kerb. The differences between the theoretical and recorded results could be related to the influence of the guardrail, to variations in the vehicle position on the bridge, and to relative conditions of the girders. Note that the strains recorded from the testing are significantly lower than those calculated from the grillage analysis.

The variation in dynamic response of the main girders in the structure was largely based on the direction of travel. As mentioned in section 3, the approach to the bridge travelling south caused the vehicles to bounce onto the first (instrumented) span because of a deteriorated road profile just before the span (Figure 3.2). Vehicles travelling north do not experience the same dynamic effects and, thus, the dynamic response of the structure to heavy vehicles travelling north is not as pronounced.

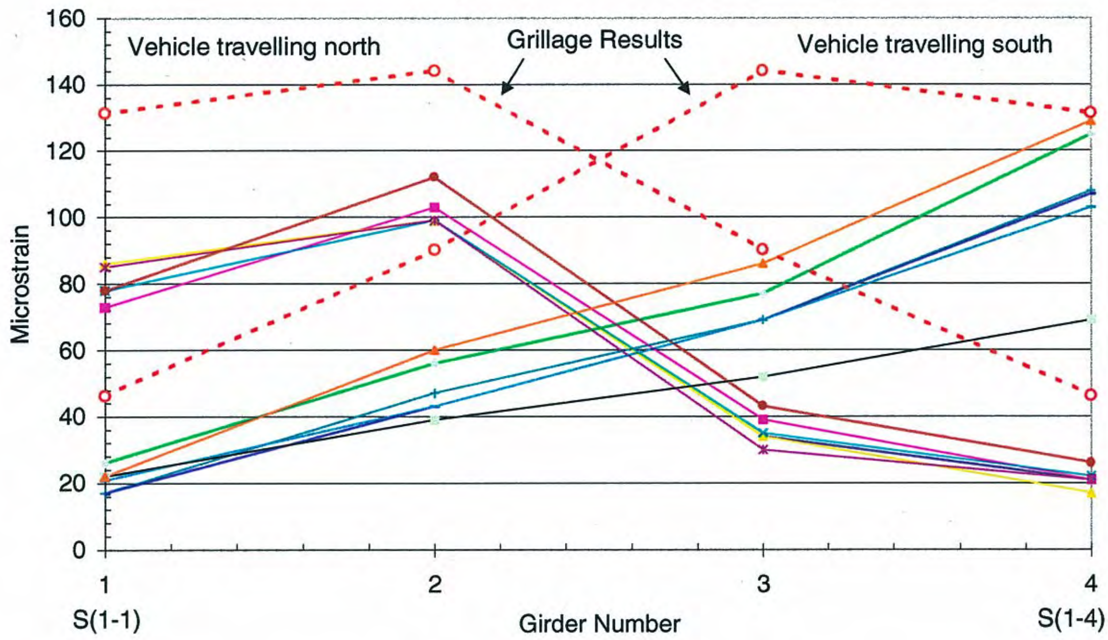


Figure 4.22 Strain distribution for known vehicle testing.

Analysis of the strain data from the known vehicle testing showed that the recorded strains were typically 10% higher for the vehicle travelling south compared with the vehicle travelling north.

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:

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

The response of the crawl test was used for the static result in the calculation of dynamic increment. The variation in dynamic increment for the known vehicle travelling south is illustrated in Figure 4.23, and these results show high dynamic responses at 60 km/h and 100 km/h. The reasons for the low value of dynamic increment at 80 km/h are a direct consequence of both the road profile and the vehicle's dynamic characteristics. The dynamic increment versus speed relationship varies from vehicle to vehicle.

The negative dynamic increment at 80 km/h is caused because the dynamic wheel forces are small (less than static) when the truck is over the instrumented span at this speed, compared with the forces at 60 and 100 km/h. Only the three transducers that were most affected by the passage of the southbound vehicle are presented in Figure 4.23. The maximum value of 28% was recorded at 100 km/h, and this value should be adopted for the dynamic increment (impact factor) for this structure.

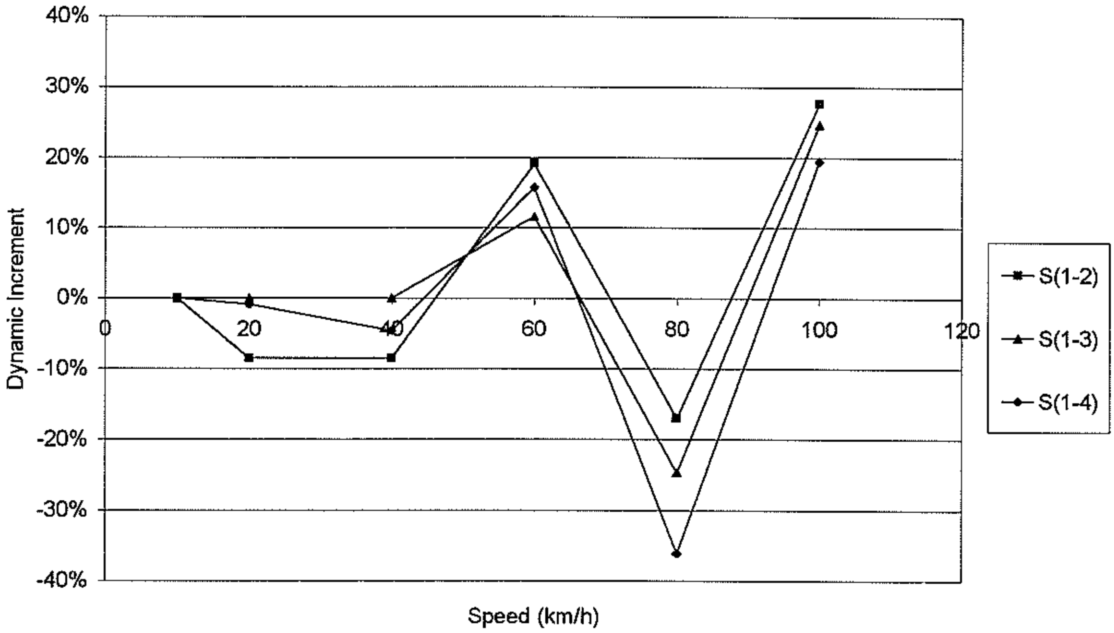


Figure 4.23 Dynamic increment plot for known vehicle travelling south.

The dynamic increment values for the northbound vehicle were well below the values illustrated in Figure 4.23 and are not presented here.

From the passage of the known vehicle, the natural frequency of the structure was found to be approximately 9 Hz. The level of damping in the superstructure is approximately 4.5 %.

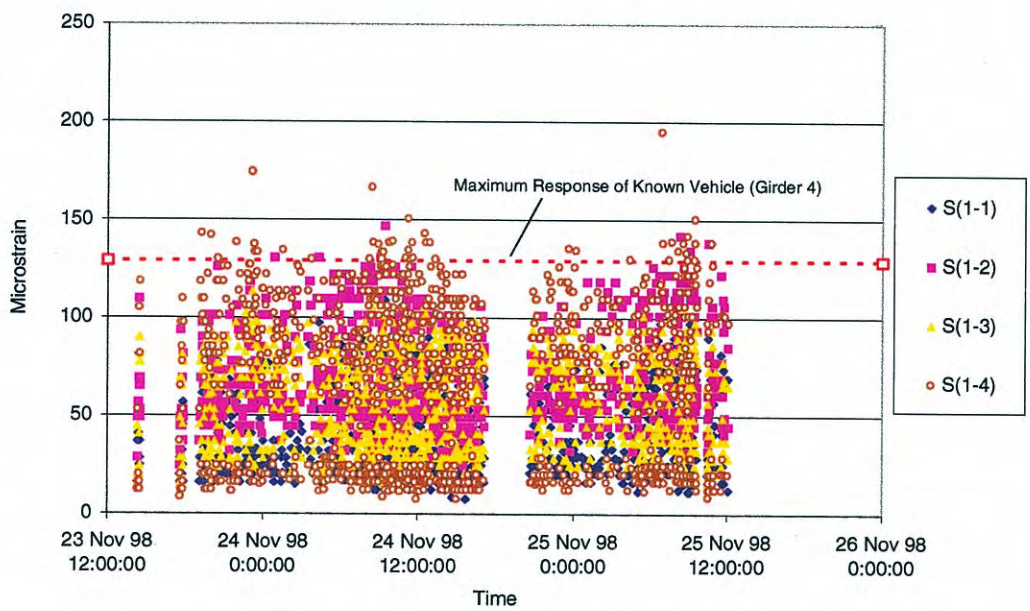
4.5 Summary

A summary of the data recorded for the Health Monitoring programme and the testing with the known vehicle is presented in Table 4.3. Figure 4.24 also illustrates a graphical comparison. In some cases (southbound traffic), the results for the maximum response of the structure to the ambient heavy vehicle traffic range up to 50% higher than the response to the known vehicle. For the northbound traffic, the response is around 30% higher than that caused by the known vehicle. Also the grillage analysis indicated that the response of the girders to the 0.85 HN posting load was 30% larger than the response to the known vehicle. Thus the response to northbound traffic is consistent with the 0.85 HN vehicle load, but the effects from the southbound traffic are approximately 20% higher than the 0.85 HN loading. The higher strains for the southbound traffic are probably related to a combination of some moderate overloading (around 10%) and the contribution of the road profile (higher impact effects).

Table 4.3 Summary of health monitoring data.

Transducer	Maximum Recorded Value (Known Vehicle)	Maximum Recorded Value (Health Monitoring)	Extrapolated Value (95% confidence limit) for 100 years
	Strain ($\mu\varepsilon$)		
S(1-1)	86	109	160
S(1-2)	112	147	220
S(1-3)	86	118	170
S(1-4)	129	195	250
SR(E)	–43	–100	–140
SR(W)	–69	–45	–75
SD(1)	43	150	220
	Deflection (mm)		
D(1-3)	2.4	3.5	4.7

Figure 4.24 Comparison of health monitoring data to the maximum response of the known vehicle.



5. Fitness for Purpose Evaluation

5.1 Main Girders

The structural assessment described in section 3.2 of this report indicated that midspan bending was the critical mode of failure for the structure. The Fitness for Purpose Evaluation of the superstructure has been determined based on this failure mode. The moment capacity available to resist the ultimate traffic live load effect was 545 kNm (ϕM-1.3DL).

For the Rakaia Bridge, there was some cracking in the girders and the demountable strain gauge transducers were installed directly over a crack in the soffit of each girder. In this bridge the measurement recorded by the transducer represents the change in crack width caused by the traffic live loads. The recorded data must therefore be adjusted based on crack-width theory in order to obtain the actual bending strain in the reinforcement in the girders.

The crack-width model is based on the ACI[2] approach as discussed in Warner et al. (1989). The maximum crack width (w_{max}) is based on the following relationship:

$$w_{max} = 0.011(hA)^{0.33}\left(\frac{D - kd}{d - kd}\right)\sigma_{st} * 10^{-3} \qquad \text{Equation (8)}$$

where:

σ_{st}	stress in the reinforcement	Parameters are:		
h	cover to bottom level of reinforcement	D	depth of section	
A	concrete tension area surrounding the reinforcing bars	d	depth to centroid of reinforcement	
		k	neutral axis parameter	

Evaluation of this formula for the girders in the Rakaia Bridge found that the recorded soffit strains should be increased by 25% to represent the strains in the reinforcement.

5.1.1 Multiple Presence

The Rakaia 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 (i.e. Multiple Presence). The probability of this occurring on an instrumented span is small and therefore it is expected that a multiple presence event would not have occurred during the monitoring period.

[2] ACI – Australian Concrete Institute

To account for multiple presence events a number of methods are available. One is to simulate a multiple presence event by summing the 95% in a 100 year event for a vehicle in each lane. This is consistent with the Bridge Manual approach 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. When applying 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 also part of this project.

Figure 5.1 summarises an assessment of the multiple presence effects on the Rakaia Bridge based on the health monitoring data, using a 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. These distributions are based on the distribution factors from the known vehicle and the extrapolated health monitoring data. The available live load capacity for the girders is also shown. The data show that approximately 73% of the girder capacity would be utilised by a multiple presence event, and that the highest strain caused by a multiple presence event is in Girder 2.

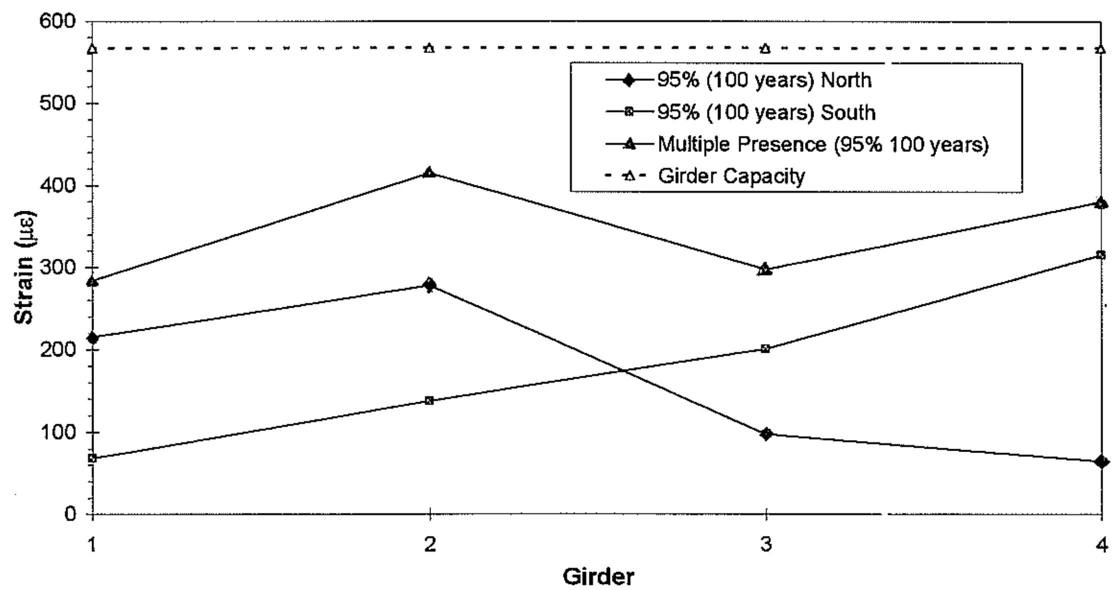


Figure 5.1 Multiple presence using the approach that is consistent with the Bridge Manual.

5.1.2 Moment versus Strain Relationship

Figure 5.2 illustrates a theoretical moment versus strain curve for a typical girder of the Rakaia Bridge. The graph presents the method used by Infratech to obtain a relationship between bending moment and strain in the reinforcement for determining the Fitness for Purpose Evaluation for this bridge. However this relationship has not been confirmed experimentally.

Line AB on Figure 5.2 represents the linear elastic behaviour of the concrete. Point B represents the point at which the concrete cracks. At this point the concrete begins to follow line BC which represents the behaviour of the concrete in the cracked state. The cracking moment is low on this bridge because of the low assumed concrete strength (17 MPa).

Because some of the girders have already cracked (under service loads), the actual relationship between moment and strain for these girders is expected to be similar to dashed line AC.

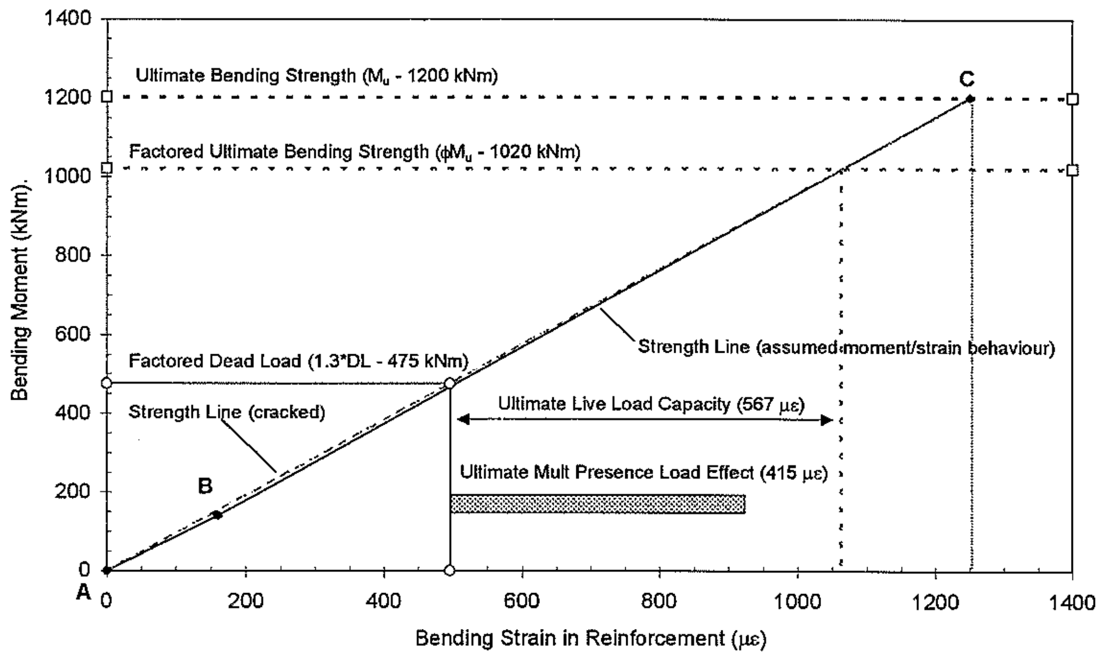


Figure 5.2 Theoretical moment versus strain relationship, and summary of Fitness for Purpose for Girder 2.

Figure 5.2 also presents the reduced capacity (ϕM) of a typical girder converted to an equivalent strain (1063 με), based on the theoretical moment versus strain relationship. The factored dead load moment (475 kNm) was converted to an equivalent strain in the reinforcement equal to 496 με. This gives an ultimate live load capacity equal to 1063 – 496 = 567 με. The strain from the multiple presence assessment (415με), which represents the ultimate traffic load effect for this bridge, is also shown on the diagram.

Table 5.1 summarises the calculation of the Fitness for Purpose Evaluation based on this data. The method for the calculation of this evaluation is outlined in section 2.4 (Equation 6), 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 137%, and this evaluation compares poorly with the HO rating evaluation (67%) and the HN posting evaluation (84%). The comparison with the HN loading is the most appropriate as this evaluation is related to ambient heavy vehicle traffic. The Fitness for Purpose Evaluation suggests that the bridge is performing much better than theoretical evaluations suggest. This is discussed in section 5.3 below.

Table 5.1 Summary of Fitness for Purpose Evaluation based on Girder 2.

Item	Result
Strength (ϕM)	1020 kNm
Factored Dead Load (*1.3)	475 kNm
Ultimate Live Load Capacity Moment ($\gamma_o R_o$)	545 kNm
Ultimate Live Load Capacity – Equivalent Strain ($\gamma_o R_o$)	567 $\mu\varepsilon$
Ultimate Traffic Load Effect (Multiple Presence)	415 $\mu\varepsilon$
Fitness for Purpose Evaluation	137%

5.2 Deck Slab

The evaluation of the deck slab capacity showed that transverse bending is determining the strength of the slab. The transverse bending capacity of the slab (ϕM) was 41 kNm which corresponds to a bending strain in the soffit of the deck slab of 260 $\mu\varepsilon$, assuming the slab behaves as a linearly elastic uncracked section. The ultimate traffic load effect for the concrete slab based on the health monitoring data is 220 $\mu\varepsilon$, and the resulting Fitness for Purpose Evaluation is 118%, or 1.18 in terms of a DCF. This is similar to the analytical evaluation based on the plate bending method. The actual failure mode may be a punching shear type failure and may give a deck capacity that is much higher than this rating.

5.3 Effect of Guardrails

The effect of the guardrails on the Rakaia Bridge is significant. The health monitoring data presented in section 4.3 indicates that the guardrails are contributing significantly to the strength of the structure. For example the maximum strain recorded in the upper rail of the guardrail was around 100 $\mu\varepsilon$. The Bridge Manual does not allow for any contribution from the strength of the guardrails to be included in the evaluation of bridges. However, load tests on bridges to failure generally indicate that these rails do contribute to the strength of the structure at high levels of load.

The Fitness for Purpose Evaluation recorded in this report has included the contribution from the guardrails, because Health Monitoring, by its definition of measuring the actual response of the structure, includes any contribution from the guardrails. It may be possible to develop a method in which the results of the Health Monitoring are modified to remove the contribution of the guardrails in the Fitness for Purpose Evaluation. This may be investigated in the later stages of this project. More extensive instrumentation of the guardrails would also be useful.

5.4 Summary

The Fitness for Purpose Evaluation for this bridge based on midspan bending of the main girders was 137%. It is significantly better than the (theoretical) posting evaluation based on the 0.85 HN vehicle load of 84% calculated by Infratech. The reasons for the differences between the evaluation obtained from Health Monitoring and that from the Bridge Manual include:

- The ambient heavy vehicle traffic induced bending moments in the bridge that are 5% to 10% higher than the 0.85 HN vehicle. This indicates that some of the traffic on this route is heavily loaded. The known vehicle was not loaded to the legal limit for this bridge and, as expected, it induced effects which were approximately 75% of the 0.85 HN vehicle.

- Comparison of the theoretical response of the bridge obtained from the grillage analysis with the recorded response for the known vehicle, shows that the recorded strains were only 50% to 60% of the theoretical strains. This is related to a number of effects, including the contribution of the guardrails which were not included in the grillage analysis, and possibly some bearing-restraint continuity effects.

- The assumed concrete strength and Young's Modulus values for this structure were also low.

The Fitness for Purpose of the deck based on bending effects is 118%, or 1.18 in terms of a DCF. The Fitness for Purpose Evaluation for the girders and the deck, recorded in this report, indicates that the structure is safely carrying the heavy vehicle loads using this route.

5.5 Comments

In the theoretical evaluation of the bridge using the grillage analysis, the edge girder was critical because the strength of the guardrail had not been included and the vehicles were positioned at the maximum eccentricity close to the edge of the bridge.

In the Health Monitoring evaluations, one of the middle girders was critical. This is probably caused by load being transferred from the edge girders to the guardrails, thus producing lower measured strains in the edge girders.

6. Conclusions

This report presents the details and results of the Health Monitoring programme and Fitness for Purpose Evaluation that apply to the Rakaia Bridge.

Theoretical Analysis
The theoretical analysis of the bridge found that midspan bending of the main girders and the bending capacity of the deck were the critical issues associated with the performance of the bridge. The rating evaluation was 67% and the posting evaluation was 84%. These values compare well with the value of 76% obtained from the TNZ Structural Inventory. According to normal practice this bridge may need to be posted based on this assessment. The DCF was 0.97 based on the plate bending method.

Health Monitoring Results
The Health Monitoring investigation found that:

- The guardrails are contributing to the strength of the bridge and have been included in the Health Monitoring evaluation of this bridge.
- The ambient heavy vehicle traffic is inducing bending moments in the bridge that are 5% to 10% higher than the 0.85 HN vehicle. This indicates that some of the traffic using this route is heavily loaded.
- The recorded strains in the girders are approximately 50% to 60% of the theoretically predicted strains. This is related to a number of effects including the contribution of the guardrails to bridge strength, which was not included in the theoretical analysis, and possibly to some bearing-restraint continuity effects.
- The uneven road profile on the northern approach to the bridge is increasing the dynamic effects on the monitored span. The dynamic increment for southbound traffic is significantly higher than that for northbound traffic.

  The highest measured impact factor for this bridge was 28%. This is similar to the impact factor of 1.3 used to determine the load rating as detailed in the Bridge Manual. Improvement of the road profile on this bridge may reduce the effects of heavy vehicles on it.

Fitness for Purpose Evaluation
The Fitness for Purpose Evaluation for the Rakaia Bridge, based on the critical midspan bending of the main girders, was 137%.

The Fitness for Purpose Evaluation for the deck was 118%, or 1.18 in terms of the DCF.

This evaluation indicates that the bridge is safely carrying the heavy vehicle traffic currently using this route.

7. Recommendations

The recommendations obtained from this investigation are for:

- Further investigation of the contribution of the guardrails to the strength of this bridge, and extrapolating the results to other bridges. Substantial economic benefit may result from including the contribution of guardrails to bridge capacity.
- Investigation of the performance of Girder 4 by increasing the monitoring period (to possibly one month).
- Consideration of whether the Rakaia Bridge is to be posted with a load limit. Although the theoretical rating suggests that the bridge should be posted, the Health Monitoring indicates that it is not required.

8. References

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