Health Monitoring of Superstructures of New Zealand Road Bridges:

Atiamuri Bridge, Waikato

Transfund New Zealand Research Report No.173

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Atiamuri Bridge, Waikato

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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 Atiamuri Bridge, on State Highway 1N, crosses the Waikato River between Tokoroa and Taupo, North Island, New Zealand. This bridge was selected as one of the ten to be health monitored, because it has a relatively unique form of construction, and has a history of load-induced problems.

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 health monitoring data. This assessment and evaluation considered bending and shear in the stringers and the bending strength of the cross girders. The evaluation excludes the supporting arch and the substructure.

Theoretical Analysis

The theoretical analysis of the bridge found that midspan bending of the stringers was the governing parameter associated with the performance of the bridge. Also the cross girders had limited reserve capacity. The calculated capacity of the stringers was based on assuming partial shear connection between the stringers and the deck, in accordance with the Transit New Zealand Bridge Manual (TNZ 1994) method. The Class is 104% according to the 1999 TNZ Structural Inventory, and the Deck Capacity Factor is 1.02.

The theoretical assessment of the superstructure of the bridge made by Infratech Systems & Services gave a rating evaluation (0.85 HO* + 0.85 HN* loading) of 90%. The cross girders were also assessed, and their rating evaluation was 95%.

Health Monitoring Results

The results of the Health Monitoring for Atiamuri Bridge show that :

- The central stringers are subject to greater stress than the edge stringers;
- Strains recorded in Stringer 3 were significantly greater than those in Stringer 2 for equivalent known events. The slip measurements on Stringer 3 also indicate significant breakdown in the composite action of this stringer;
- The ambient heavy vehicle traffic population produced structural responses that were significantly greater than the response produced by a known heavy vehicle.

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* HO Highway overweight vehicles; HN Highway normal vehicles

- The response of the abutment cross girders to ambient traffic is greater than expected. This may be the result of increased dynamic activity at the bridge abutment interface;
- The natural frequency of the bridge was difficult to determine, but one of the dominant frequencies recorded was 15 Hz, and this was associated with stringer response;
- The maximum dynamic increment recorded for the bridge was 35%.

Fitness for Purpose Evaluation

The Fitness for Purpose Evaluation for this bridge based on midspan bending of the stringers was 80%. However the theoretical posting evaluation for the bridge is 105%.

This difference (of 25%) suggests that the bridge is not performing as well as might be expected based on theoretical calculations. It is likely to be the result of greater than expected deterioration in the composite action between the deck and the stringers.

The Fitness for Purpose Evaluation of the cross girders, based on Health Monitoring, was 106%. This is acceptable and suggests that stringer performance (rather than the cross girders) is the critical issue with respect to bridge deck capacity.

Recommendations:

The following recommendations are made:

- Undertake further investigations into the extent of composite action mobilisation on the Atiamuri Bridge.
- Monitor the composite action, and the associated deterioration, of the Atiamuri Bridge, to gain a better understanding of the phenomenon, because it will be relevant to other New Zealand bridges.
- As long-term monitoring of the Atiamuri Bridge is also part of the project, decisions regarding this issue await the long-term results.

A correctly designed Health Monitoring programme could achieve the following objectives:

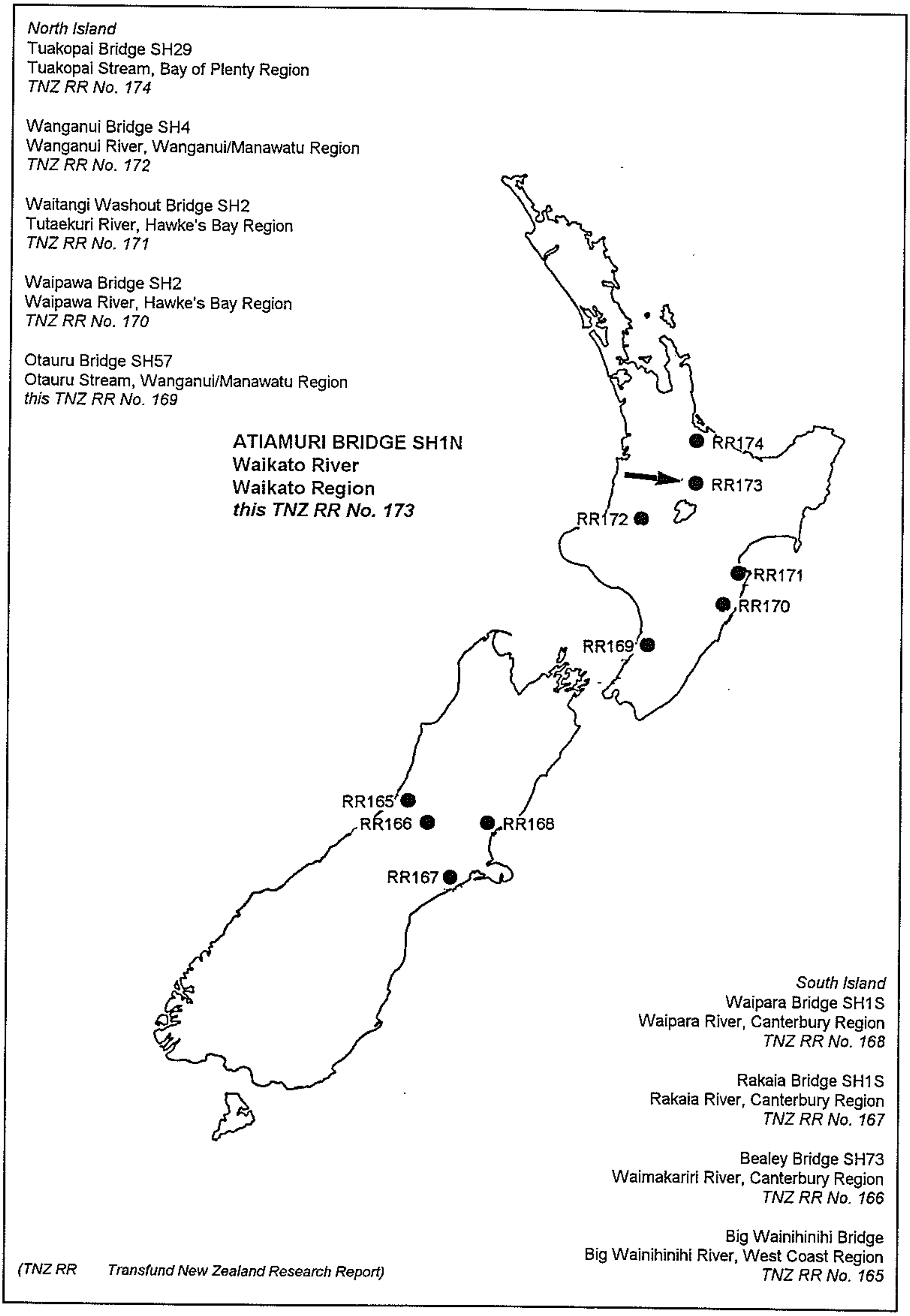
- Manage the risk of failure at the Atiamuri Bridge by continuous monitoring.
- Obtain an improved understanding of this deterioration phenomenon that can be used to manage the New Zealand bridge population.
- Determine the most appropriate rehabilitation strategy to ensure that maximum service life is obtained from the existing bridge.

Abstract

Bridge Health Monitoring is a method for 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 Atiamuri Bridge, on State Highway 1N, crosses the Waikato River between Tokoroa and Taupo, North Island, New Zealand. It was selected as one of the ten to be health monitored because it has a relatively unique form of construction, with a steel–concrete composite deck system, and has a history of load-induced problems.

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



1. Introduction

1.1 Health Monitoring of Bridges

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, and 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 Atiamuri Bridge over the Waikato River, on State Highway 1N (SH 1N) between Tokoroa and Taupo, in Waikato Region, North Island of New Zealand. The bridge was chosen because it has a unique design with a history of load-induced problems.

The objective of this investigation was to evaluate the Fitness for Purpose of the superstructure of the Atiamuri 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.
- Subsequently, the Health Monitoring evaluation was compared with the conventional rating.

The critical parameters associated with this Fitness for Purpose Evaluation were:

- Midspan bending strength of the main steel stringers;
- Shear strength of the main steel stringers;
- Midspan bending strength of the steel cross girders.

The Fitness for Purpose Evaluation presented in this report is based only on the above components and does not account for any part of the substructure, or the supporting arch.

This bridge is also being health monitored on a long-term basis for a 12-month period as part of this project to investigate:

- The appropriate length of time to health monitor a structure, including the cost-benefit trade-off.
- Appropriate techniques for evaluating multiple presence effects from health monitoring data.
- Variations in traffic characteristics over a long period of time.

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.

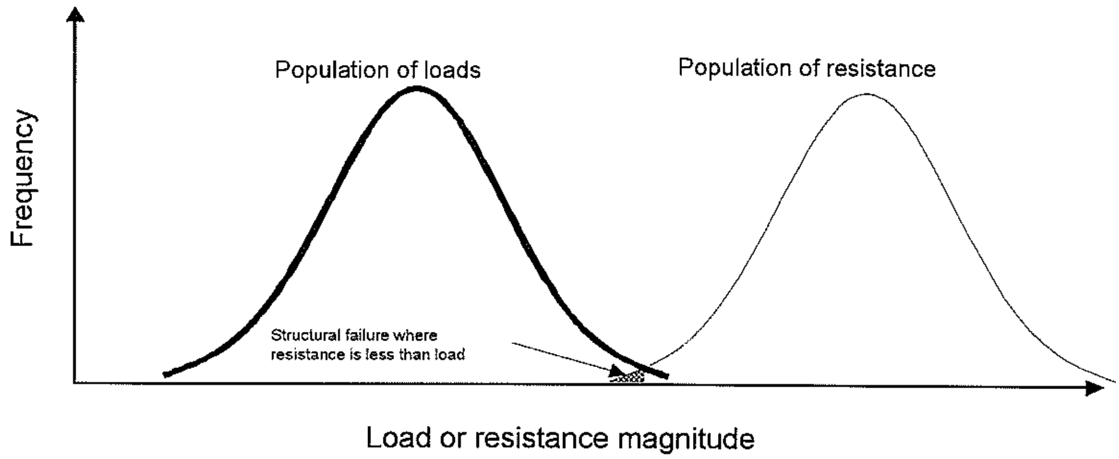


Figure 2.1 Statistical representation of structural failure.

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 (compared with typical design scenarios).

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. In some cases, uncertainty may be reduced by testing all or part of the structure. 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 often little 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 that is 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 the Bridge 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 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

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;
- A more detailed knowledge of the transport task to which the bridge is subjected.

Behavioural tests using a known vehicle were conducted on the Atiamuri 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 Atiamuri 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 Atiamuri Bridge is located on SH 1N where it crosses the Waikato River, and is a twin-cantilevered steel-truss structure hinged at the centre of the centre span. The span arrangement is 27.4 m/54.9 m/27.4 m. The deck comprises a cast in-situ reinforced-slab composite with four longitudinal stringers, spanning 5.5 m between steel cross girders. Construction of the bridge was completed in 1957 and the structure is illustrated in Figure 3.1.

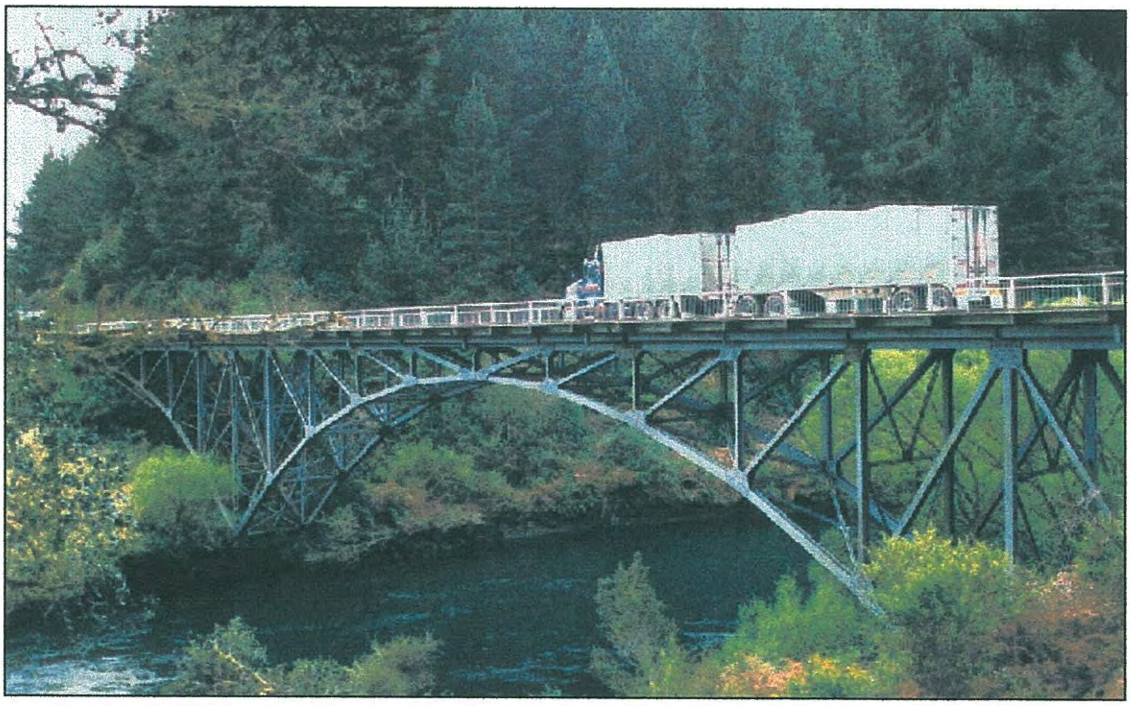


Figure 3.1 Atiamuri Bridge over the Waikato River.

The structure has a history of vehicle load-induced problems (Works Consultancy Services (WCS) 1993). These include:

- Cracking of the cross girder webs at the stringer transom joint;
- Possible fatigue issues with some of the main truss members;
- Fatigue-induced problems with the cross girders requiring repair;
- Inadequate shear connection between the steel stringers and the concrete deck.

This investigation focuses mainly on investigating the behaviour and capacity of the steel stringers, although the cross girders are also considered. The WCS (1993) report recommended that further investigations should be conducted into the problems associated with the stringers. The other issues that were identified have not been investigated in this report. However it is feasible to investigate some of these problems using Health Monitoring techniques.

The current theoretical load rating of the bridge in the Transit New Zealand Structural Inventory (1999) is:

- Bridge Class 104%
- Deck Capacity Factor (DCF) 1.02

These ratings are based on the evaluation methods set out in Section 6 of the Bridge Manual, which are described 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 of responses to 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, and were confirmed by on-site measurements.

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

- Concrete Deck f'_c =. 21 MPa, E = 22 100 MPa
- Steel Members f_y =.230 MPa, E = 200 000 MPa

3.2.1 Stringer Bending

A summary of the maximum bending moments 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 the stringer with the vehicle at the greatest allowable eccentricity. The critical stringers identified by the grillage analysis were the central stringers. The capacity of these stringers is less than the edge stringers because of the existence of the large kerbs. These tend to provide additional capacity to the edge stringers.

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

Table 3.1 shows that the 0.85 HO combined with the 0.85 HN vehicle caused the maximum response (165 kNm). The maximum bending moment in a typical central stringer due to the dead load is 45 kNm.

Table 3.1 Results of grillage analysis for midspan bending moment (kNm) in a typical central stringer.

Load	Bending Moment (kNm)
Dead Load	45
Known Vehicle	60
2x 0.85HN Vehicles (Posting Load)	110
0.85HO + 0.85HN Vehicles (Rating Load)	165

The shear connection between the stringers and the deck is not sufficient to ensure full composite action between the deck slab and the steel stringers. The WCS (1993) report also noted that the shear connectors were over-stressed. This is due to a practice at the time (1950s) of the design of the bridge to allow bond stress to contribute to composite action. That report also notes that the fatigue life of the shear connector has been exceeded and that indications of relative movement between the stringer and deck slab have been recorded.

In this section 3, the capacity of the steel stringers has been based on the assumption that the shear connectors are still providing partial composite action. This assumption will be reviewed later in this report. Based on this assumption, the bending capacity of the partially composite steel stringers of the superstructure was calculated in accordance with Section 13 of the Steel Structures Standard (NZS 3404: Part 1 1997). On this basis, the ultimate moment capacity (ϕM) is 340 kNm.

3.2.2 Stringer Shear

The vertical shear force at the supports in a typical central stringer, obtained from the grillage analysis is presented in Table 3.2. The shear capacity (ϕV) of a typical steel web was found, in accordance with Section 13 and Section 5 of the Steel Structures Standard (NZS 3404: Part 1 1997), to be 380 kN.

Table 3.2 Results of grillage analysis for shear (kN) in a typical central stringer.

Load	Shear Force (kN)
Dead Load	30
Known Vehicle	45
2x 0.85HN Vehicles (Posting Load)	70
0.85HO + 0.85HN Vehicles (Rating Load)	110

3.2.3 Cross Girder Bending

A summary of the maximum bending moments resulting from the various loads is presented in Table 3.3. The results in the table are not factored, and they represent the maximum bending moment in an internal cross girder when vehicles are at the most adverse eccentricity. Table 3.3 shows that the 0.85 HO vehicle in the centre of the carriageway caused the maximum response (195 kNm). The 0.85 HO + 0.85 HN (140 kNm) loading was less critical because the cross-girder end cantilevers transfer moment away from the centre of the cross girder when load is applied to the cantilever ends. The bending moment in the centre of the cross girder under dead load is 5 kNm.

Table 3.3 Results of grillage analysis for midspan bending moment (kNm) in a typical cross girder.

Load	Bending Moment (kNm)
Dead Load	5
Known Vehicle	61
2x 0.85HN Vehicles (Posting Load)	116
0.85HO + 0.85HN Vehicles (Rating Load)	140
0.85HO Vehicle in centre of bridge (Rating Load)	195

The ultimate moment capacity of the cross girder (ϕM) is 366 kNm (assuming partial shear connection between the deck and the stringers). Shear capacity was checked and found to be adequate (i.e. has a large reserve capacity), so was not considered further.

3.2.4 Deck Capacity

The capacity of the concrete deck was not considered in this report. A visual inspection also indicated that the concrete deck is in good condition. The load rating of the deck, obtained from the TNZ Structural Inventory, indicates a Deck Capacity Factor equal to 1.02.

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. The results of the theoretical load evaluation of the structure are presented in Table 3.4. It has been assessed for bending and shear in the stringers, and for bending in the cross girder. The table also presents a comparison of the load ratings calculated by Infratech Systems & Services (Infratech), with that found in the TNZ Structural Inventory.

A value of 1.3 was used for the impact factor, and a value of 1.3 was used for the dead load factor in calculating the load ratings. The impact factor is not included in the values of moment and shear presented in Table 3.4, but the impact factor has been included on the ratings and postings (percentage values).

Table 3.4 Summary of theoretical load evaluations for the bridge superstructure.

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)
Stringer Bending	340kNm	165kNm	110kNm	45kNm	90%	105%	
Stringer Shear	380kN	110kN	70kN	30kN	160%	195%	104%
Cross-girder Bending	366kNm	195kNm	116kNm	5kNm	95%	125%	

The overall rating of the superstructure is taken as the minimum rating of all the components. For this bridge, the mid-span stringer bending is the critical failure mode with a rating of 90%. This compares reasonably well with the rating of 104% which is documented in the TNZ Structural Inventory. The difference may be related to assumptions regarding the degree of composite action. While the cross-girder capacity does not control the bridge rating, its rating is similar to the stringer rating. If different assumptions were made with respect to composite action between the stringers and the deck, the cross girder capacity could become critical and control the bridge rating.

3.4 Summary

The Atiamuri Bridge, in Waikato Region, was analysed using a grillage analysis to determine the bending moment and shear in the stringers and the bending moment in the cross girders of a typical span, based on various vehicle loadings.

The bending moment in the stringers was found to govern the strength and therefore it determines the rating of the superstructure. However, this rating is based on the assumption that partial composite action exists between the stringers and deck. While the cross girders were not critical, they had a similar rating to the stringers. Hence cross-girder capacity could control the bridge rating as the composite action was slightly better that that assumed in this analysis. Therefore, while the remainder of the superstructure was not analysed, the stringers are believed to be the critical component in determining the load rating of the structure.

The Deck Capacity Factor is 1.02 in the TNZ Structural Inventory. A visual inspection also indicated that the deck is in good condition. Therefore the evaluation of the deck is not considered further in this report.

Based on the results from this analysis, the Health Monitoring programme concentrated on evaluating the Fitness for Purpose for the stringers based on midspan bending. The Fitness for Purpose for the cross girders (based on midspan bending) was also investigated.

4. Health Monitoring Programme

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

- Short-term health monitoring of the ambient heavy vehicle traffic for a period of approximately two 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 Atiamuri Bridge over the Waikato River.

4.1 Instrumentation

The instrumentation installed on the bridge included five Demountable Strain Gauge transducers (S) and three Foil Strain Gauge transducers (F). Figure 4.1 illustrates the locations of the four transducers installed at the midspan of the four stringers (S1 to S4) in the first segment of the structure. The first two cross girders (CG) and two of the bottom chords (BC) of the truss were also instrumented. The positions of the instrumented segments are illustrated in Figure 4.2.

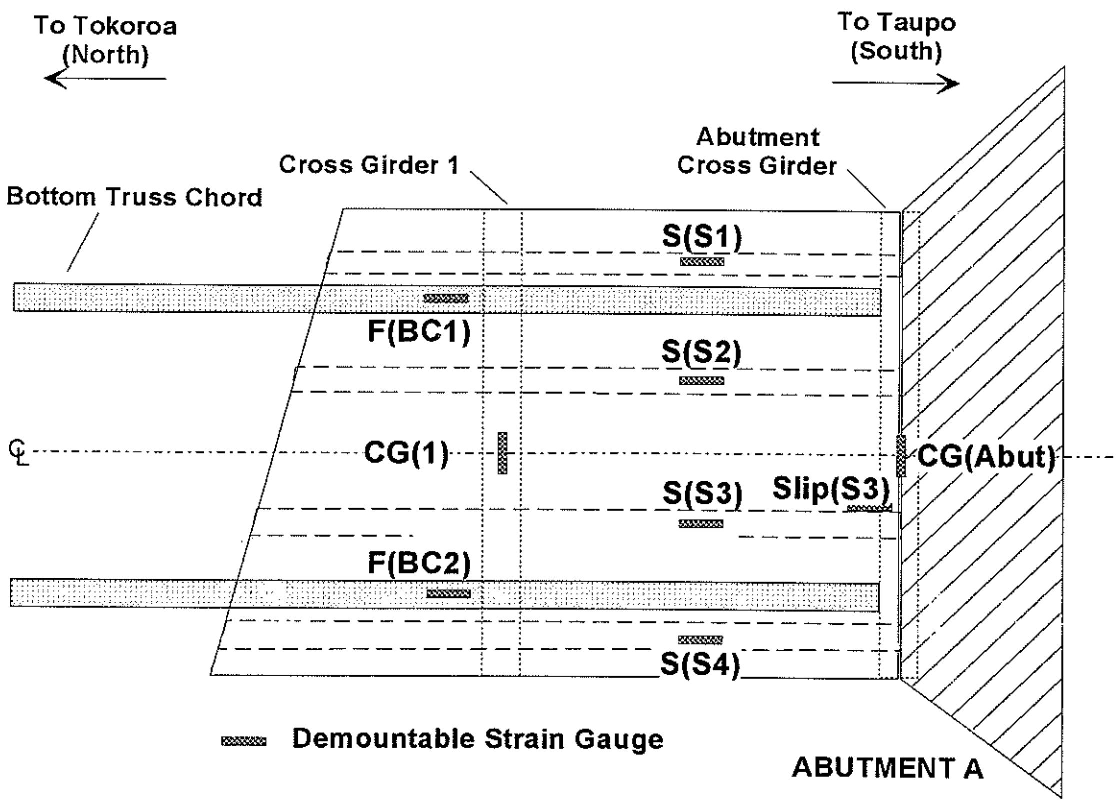


Figure 4.1 Instrumentation plan for Atiamuri Bridge.

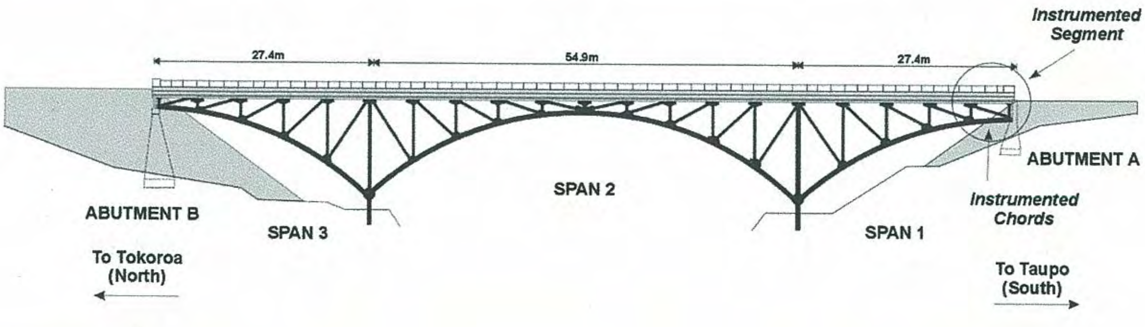


Figure 4.2 Cross section of the Atiamuri Bridge showing the position of the instrumented segment of the truss.

After testing with the known vehicle, transducer CG(Abut) was removed from the abutment cross girder and installed between Stringer 3 and the concrete deck above the cross girder. This new location was used to determine the slip between the steel stringer and the concrete deck directly above it (Figure 4.3). The amount of slip is directly related to the degree of composite action occurring between the stringer and the concrete deck. The new transducer position was named Slip(S3) and is illustrated in Figure 4.1.

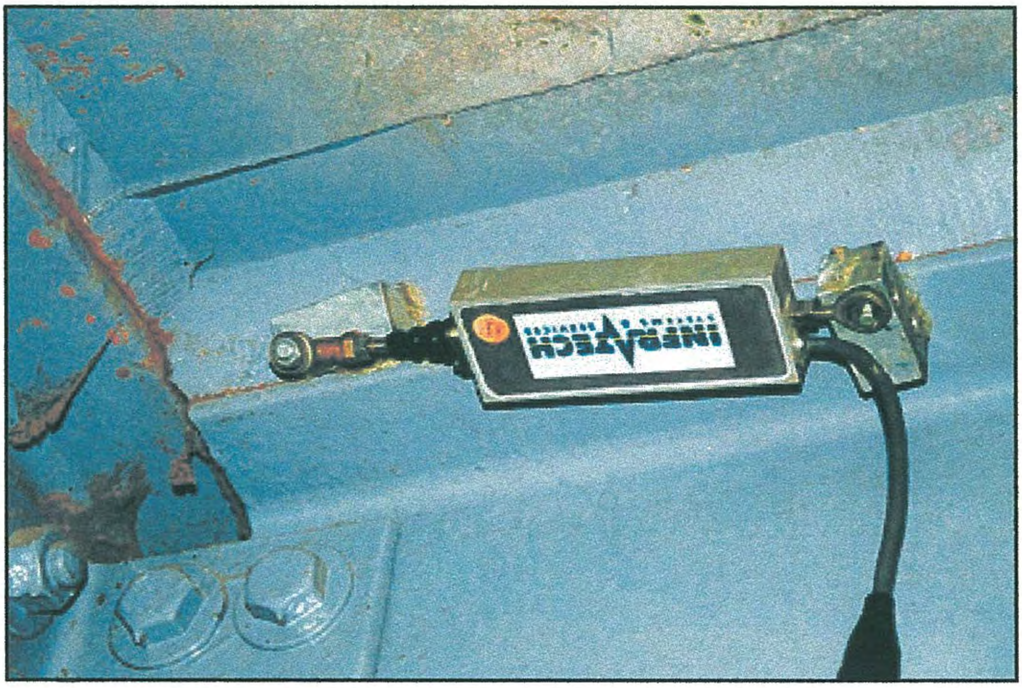


Figure 4.3 Demountable strain gauge installed to record the slip between steel stringer and concrete deck.

Figure 4.4 shows the installation of a demountable strain gauge on the abutment cross girder. The demountable strain gauges were installed using clamps or glue onto the steel and concrete, to ensure that the transducers did not move under vibrations caused by heavy vehicle traffic.

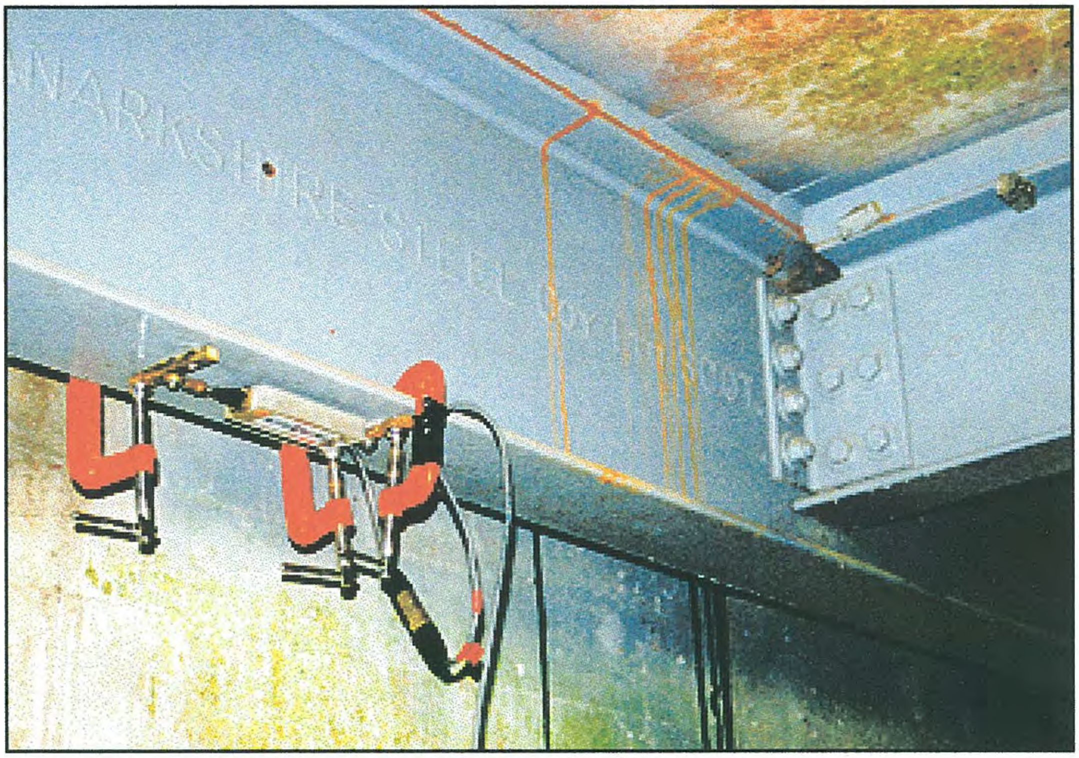


Figure 4.4 Instrumentation on the abutment cross girder by Stringer 3.

The demountable strain gauges (gauge length 230 mm) used on the stringers measured strain at a point 20 mm below the soffit. The results have been corrected to represent the strain in the soffit of the stringers. The sign conventions used throughout this report include positive values for tension strains, and negative values for compression strains.

4.2 Procedure

The Health Monitoring of the structure began on Thursday 1 October, and continued until Saturday 3 October, 1998, giving a total monitoring period of approximately 40 hours. During the monitoring period, the response of the bridge to 934 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 Friday 2 October, 1998. The vehicle used for the testing (shown on the bridge in Figure 3.1) was supplied by TD Haulage Ltd. It was a seven-axled heavy vehicle of known mass (total axle mass of 43.9 tonnes) and dimensions, which are illustrated in Figure 4.5.

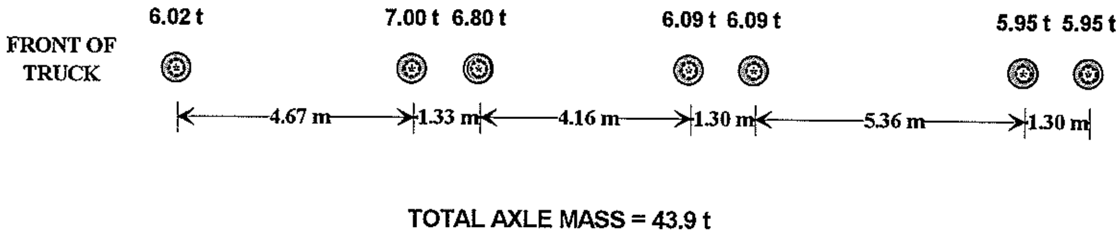


Figure 4.5 Axle mass and configuration of the known vehicle used for behavioural testing.

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 different speeds. The tests were conducted with the vehicle travelling in both directions (south, then north) from a crawl (20 km/h) to 80 km/h, in increments of 10 km/h. The lateral position of the known vehicle was in the normal lane. 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 the test runs with the known vehicle.

4.3 Short-Term Health Monitoring Results

4.3.1 Stringer Response

A typical strain response versus time was graphed (as waveforms in Figure 4.6) for the midspan bending strains recorded during the health monitoring for the passage of a typical heavy vehicle. The waveforms show a significant free vibration response after the vehicles have passed over the instrumented segment. This waveform represents a vehicle travelling towards Tokoroa (north) which means that the instrumented span is the first span that the vehicle crosses.

A scatter diagram represents the maximum strains recorded during the passage of each heavy vehicle for the entire Health Monitoring period. Figure 4.7 presents the scatter diagram for the midspan bending strains in the four stringers. 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 scatter diagram (Figure 4.7) displays consistently higher responses for transducer S(S3). This may indicate that higher strains are being recorded in this stringer or that the vehicles travelling north are more heavily loaded. This will be discussed further in section 4.4.

Figure 4.6 Response versus time for midspan bending strain transducers on the stringers for event occurring at 14.58, 2 October 1998 (vehicle travelling north to Tokoroa).

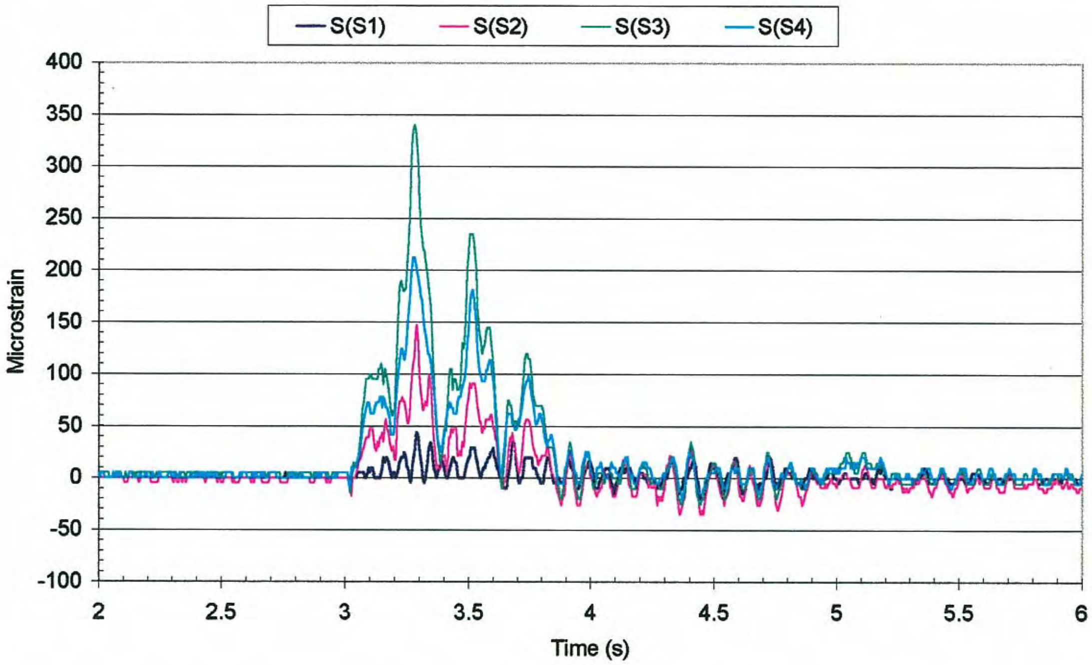
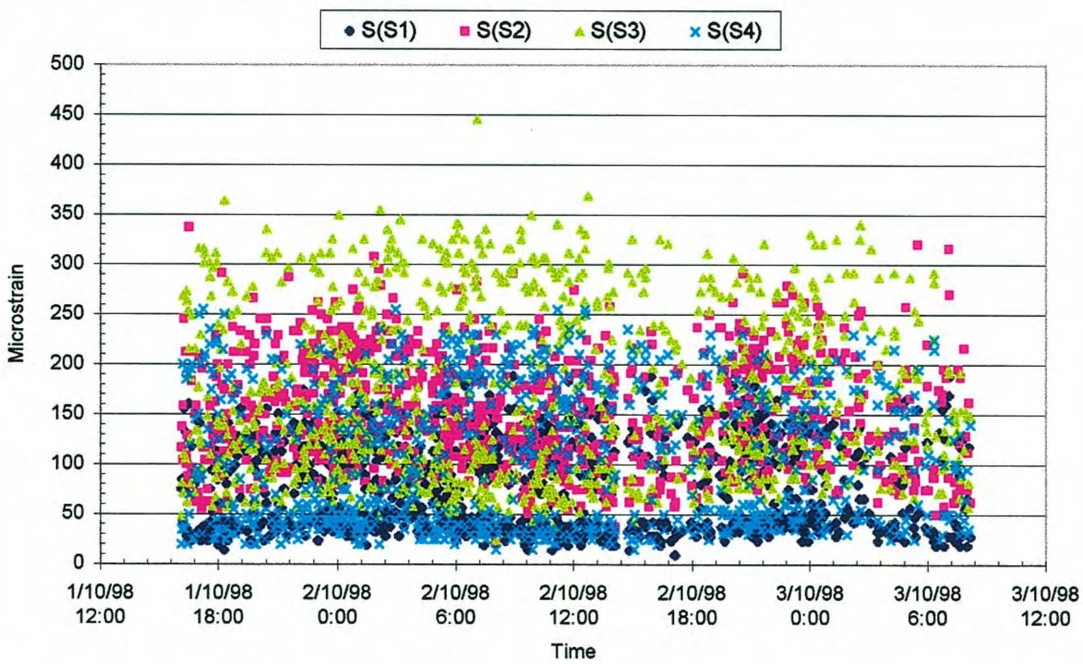


Figure 4.7 Scatter diagram for midspan bending strains in the stringers.



The response of the transducer measuring the relative movement (slip) between Stringer 3 and the concrete deck is presented in Figure 4.8. The large jump in the response of the transducer is due to the sudden application of vehicle load from a vehicle travelling north. This response would be reversed horizontally for a vehicle travelling in the opposite direction. The maximum strain of 1230 με corresponds to a movement or slip of approximately 0.28 mm between the steel girder and the concrete deck slab.

The large movement (slip) between the steel stringer and the concrete deck indicates that the composite action between the two components has been reduced. The scatter diagram for transducer Slip(S3) is presented in Figure 4.9, and it indicates that the transducer has a large tensile and compressive response to the passage of the heavy vehicles.

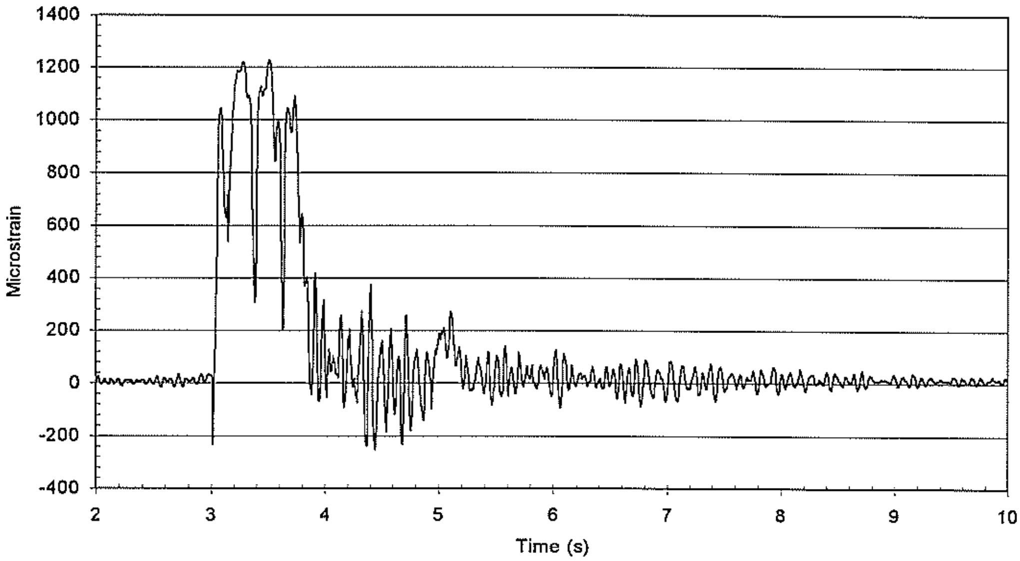


Figure 4.8 Waveform for transducer Slip(S3) showing slip between Stringer 3 and deck, for event occurring at 14:58, 2 Oct 1998 (vehicle travelling north towards Tokoroa).

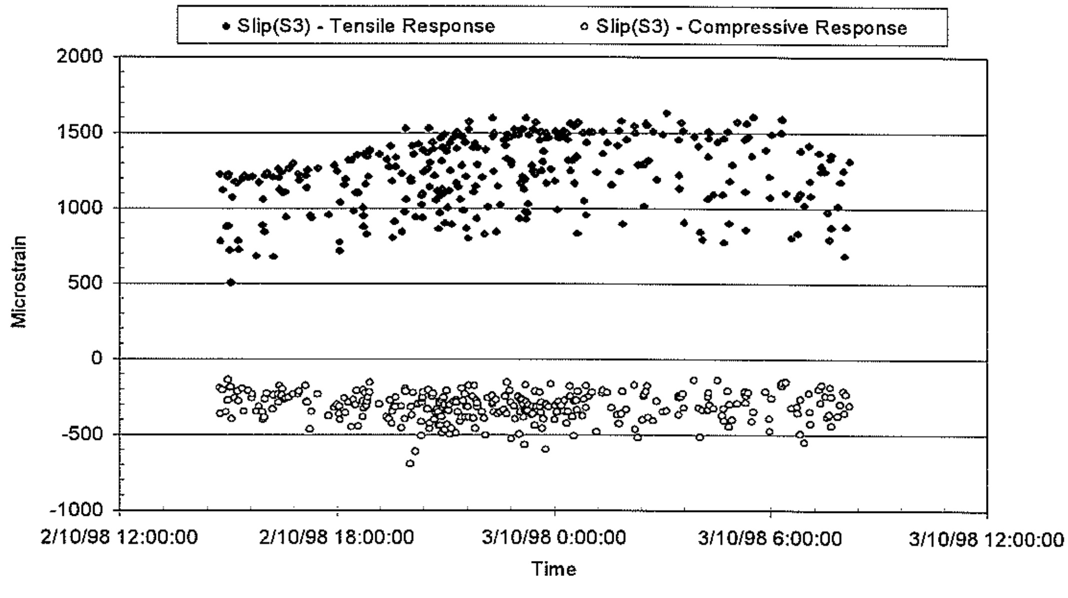


Figure 4.9 Maximum and minimum response scatter diagram for transducer Slip(S3).

4.3.2 Cross Girder Response

The response of Cross Girder 1 (abutment) to the passage of a typical heavy vehicle is illustrated in Figure 4.10. The waveform once again shows a large degree of free vibration in the cross girder after the vehicle has driven over this section of the structure.

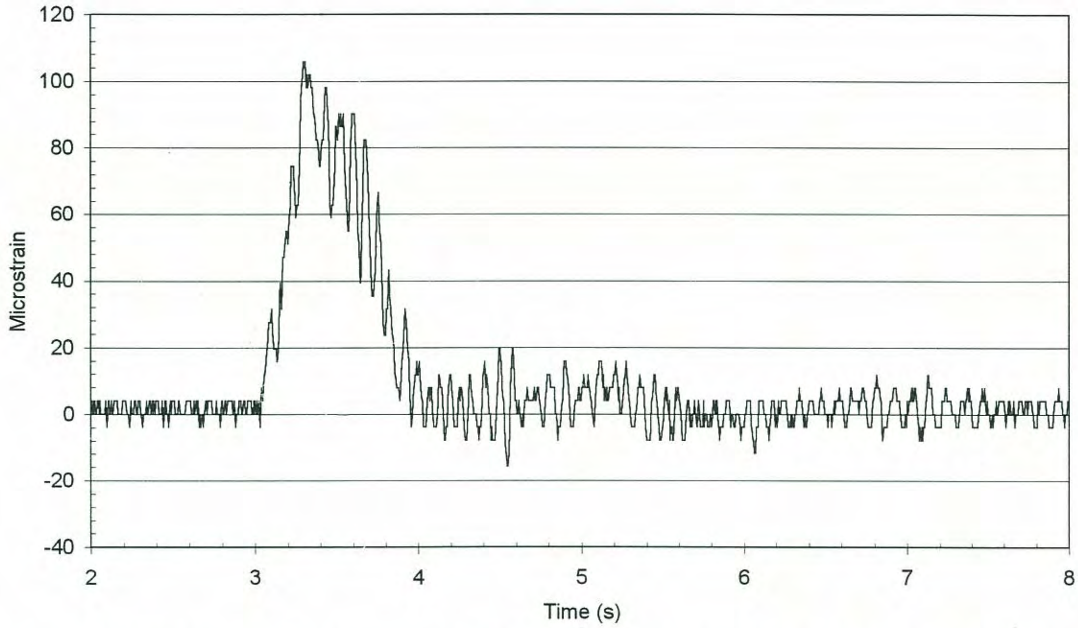


Figure 4.10 Waveform for transducer CG(1) for event occurring at 14:58, 2 Oct 1998 (vehicle travelling north towards Tokoroa).

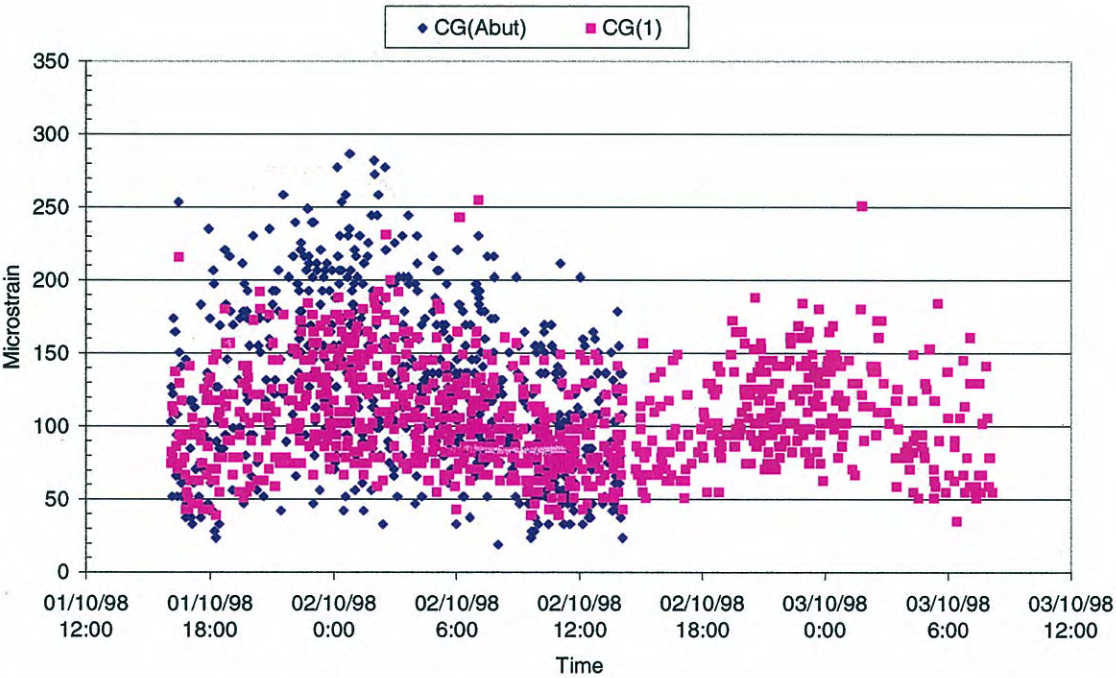


Figure 4.11 Scatter diagram for the transducers measuring strain in the cross girders.

The scatter diagram for the transducers installed on both cross girders is presented in Figure 4.11. The diagram clearly shows the point at which the monitoring of transducer CG(Abut) was stopped, at approximately 2pm on 2 October, 1998.

4.3.3 Bottom Chord Response

The response versus time for the transducers installed on the bottom chords of the truss is presented in Figure 4.12. The waveforms show a large dynamic response with the passage of the heavy vehicle. As expected the response of the bottom chords is primarily tensile, although some compression is recorded as the vehicle moves off the instrumented segment. The response is higher than in transducer F(BC2) because the vehicle is over this left side of the bridge travelling north.

The scatter diagram for the bottom chord transducers is presented in Figure 4.13. The diagram shows consistent results for the entire monitoring period.

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 is presented for transducer S(S1) in Figure 4.14. The histogram illustrates a very large number of samples corresponding to strains of less than 45 $\mu\varepsilon$. The remaining results are more normally distributed. This 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.

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 4.15. In this figure the data are separated into opposite (north and south) directions for the transducers installed on the stringers (midspan). Also 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, as outlined in section 2 of this report.

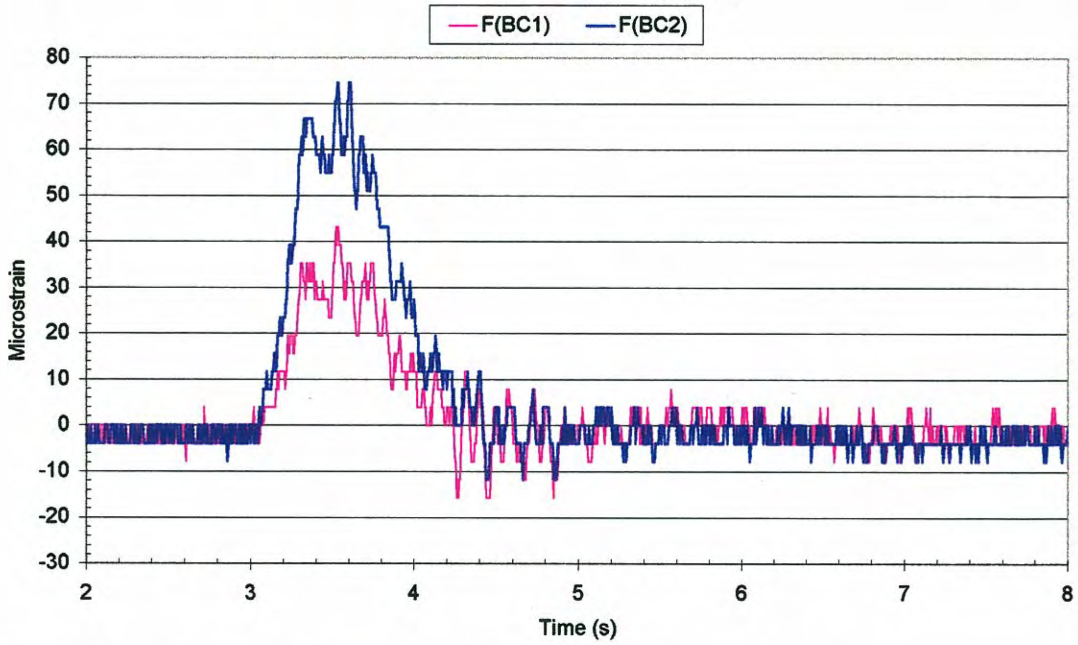


Figure 4.12 Waveforms for transducers installed on the bottom chords for event occurring at 14.58, 2 Oct 1998 (vehicle travelling north to Tokoroa).

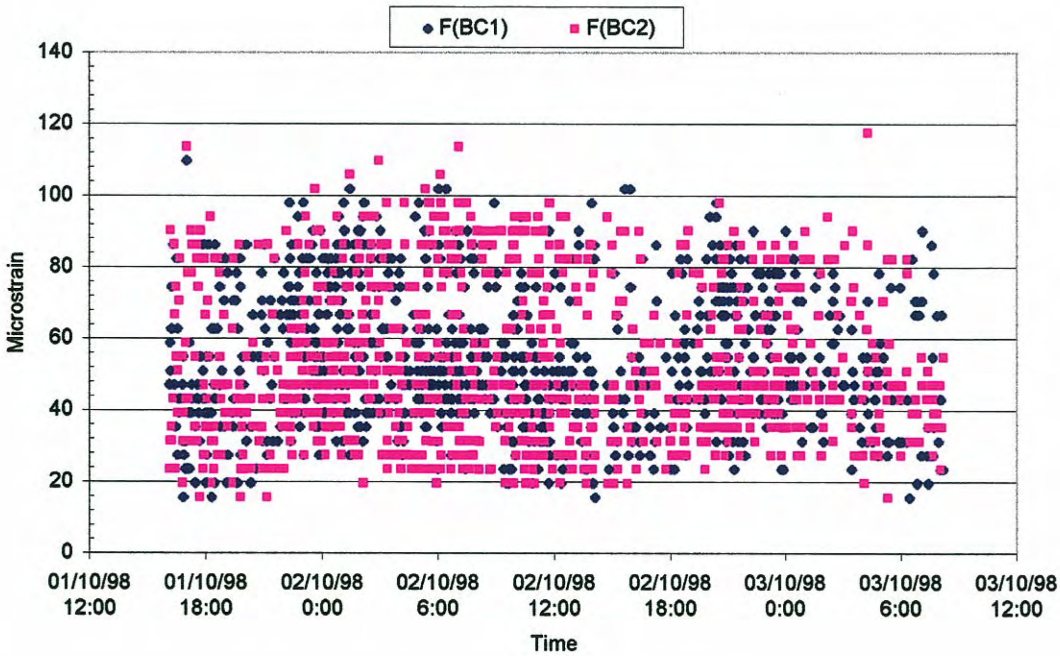


Figure 4.13 Scatter diagram for bottom chord transducers.

Figure 4.14 Histogram and cumulative distribution function for transducer S(S1).

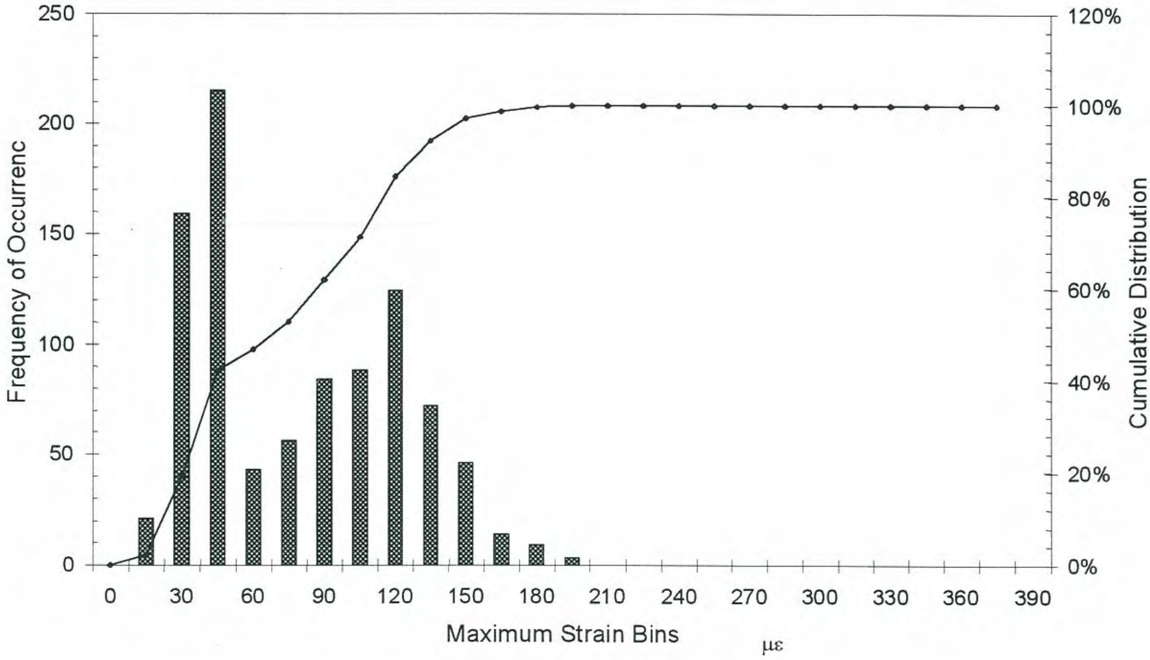
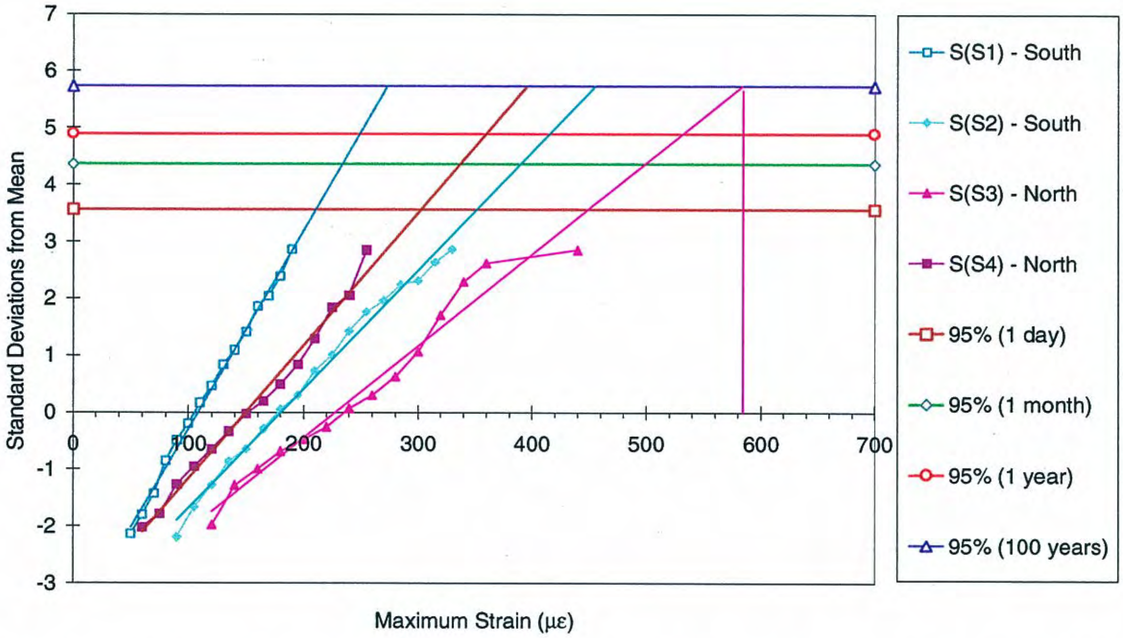


Figure 4.15 Inverse normal plot for strain transducers installed on the stringers midspan).



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 transducer S(S3) on the midspan of stringer 3. The extrapolated value is approximately 585 με.

The inverse normal plots for the transducers installed on the bottom chords are presented in Figure 4.18, in which the data for the vehicle travelling in both directions have been used.

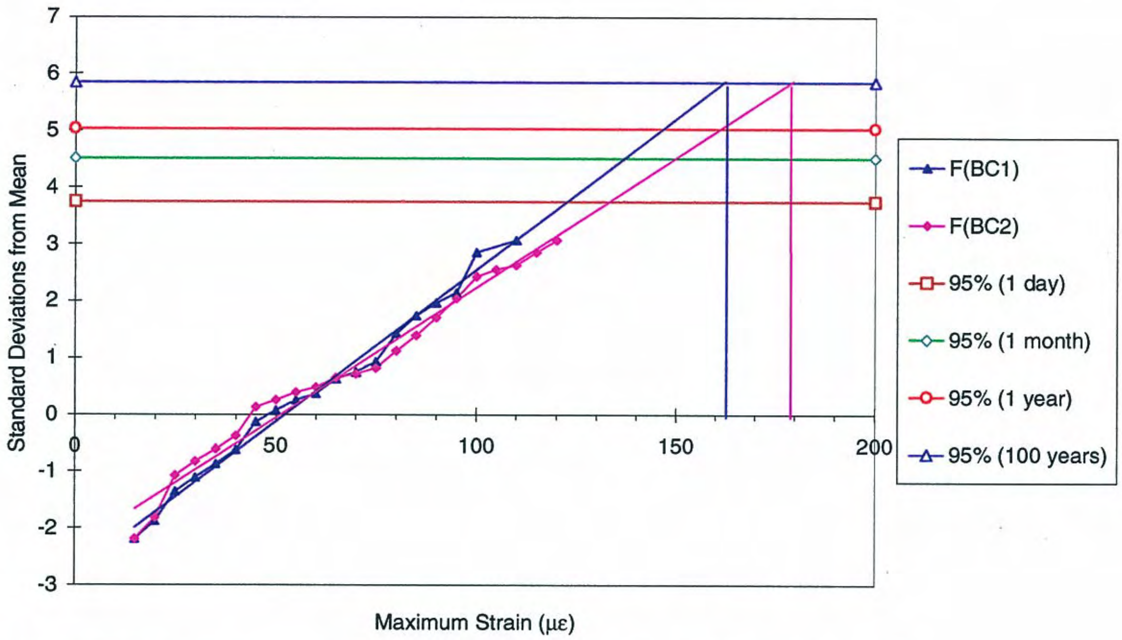


Figure 4.18 Inverse normal plot for transducers installed on bottom chords.

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 obtained from inverse normal distributions.

(from Figures 4.15-4.18)

Transducer	Maximum Recorded Value (Health Monitoring)	Extrapolated Value (95% confidence limit) for 100 years
	Strain ($\mu\varepsilon$)	
S(S1)	190	270
S(S2)	335	450
S(S3)	445	580
S(S4)	255	390
CG(Abut)	285	450
CG(1)	250	350
F(BC1)	110	165
F(BC2)	120	180
Slip(3)	1565	2000

4.4 Known Vehicle Testing

A typical waveform from the testing with the known vehicle (travelling south) is presented in Figure 4.19. The waveform shows the high level of dynamic activity caused by the vehicle crossing the bridge before it passes over the instrumented span. Some prolonged vibration of the stringers also occurs after the known vehicle has passed over the instrumented span and off the bridge. The waveform indicates the seven distinct spikes of the axles of the known vehicle as it passes over the span.

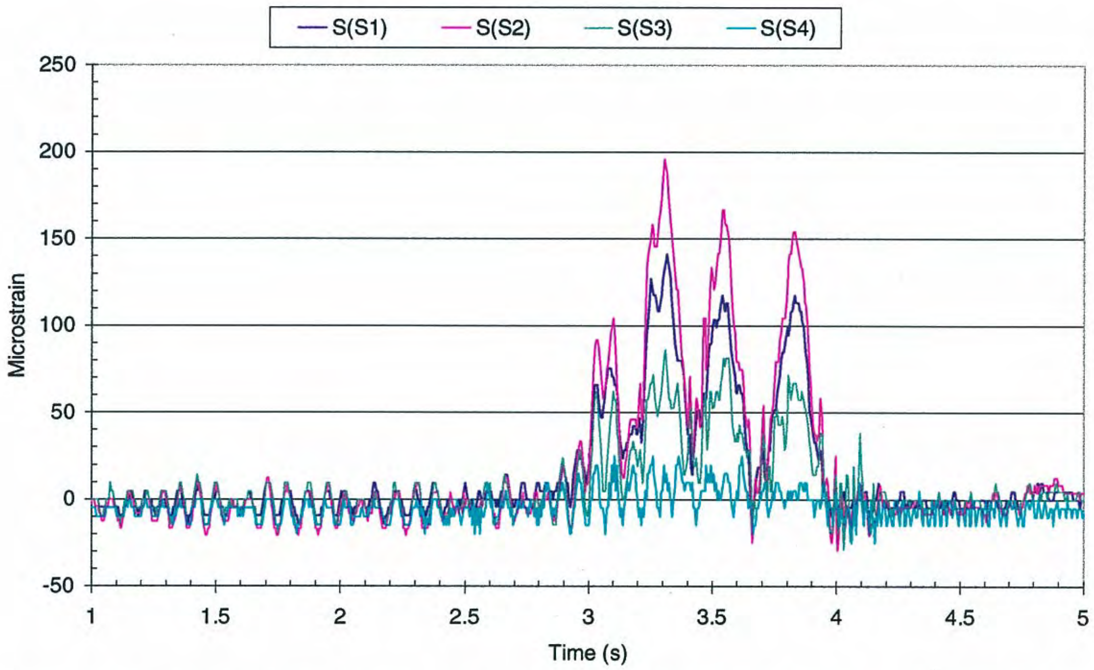


Figure 4.19 Typical waveform for the known vehicle travelling south at 80 km/h towards Taupo.

The known vehicle testing was performed at different vehicle speeds ranging from a crawl to 80 km/h. The maximum strains for each transducer recorded from the known vehicle are presented in Table 4.2.

Based on the waveform on Figure 4.19, the frequency of oscillation before the heavy vehicle reaches the stringer span is approximately 15 Hz. This before-passage frequency would not be used to determine the natural frequency of the bridge, for which the frequency of oscillation after the passage of the vehicle is normally used. For this bridge, the natural frequency is difficult to determine, and seems to be a combination of frequencies, probably resulting from a summation of global and local frequencies. This complex frequency response means that the damping associated with the bridge could not be quantified. Clearly however the damping was not particularly strong.

Table 4.2 Maximum strains recorded for known vehicle testing.

Transducer	Maximum Strain
S(S1)	160με
S(S1)	220με
S(S1)	280με
S(S1)	185με
CG(Abut)	120με
CG(1)	120με
F(BC1)	80με
F(BC2)	80με

The distribution of strain into each of the stringers from the known vehicle data is presented in Figure 4.20. The distribution presented is consistent with the data collected from Health Monitoring of the ambient heavy vehicle traffic. It shows that higher strains are being recorded in Stringer 2 for vehicles travelling south to Taupo, and for vehicles travelling north to Tokoroa the maximum strain was recorded in Stringer 3.

The figure shows that the maximum response of Stringer 3 is higher than the maximum response for Stringer 2. This indicates that Stringer 3 is attracting more strain, and not that the ambient heavy traffic travelling north is more heavily loaded. This may be caused by a difference in the degree of composite action between stringers, or by a difference in the road profiles for the two directions.

Figure 4.20 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. The position of the vehicle for the grillage was 600 mm out from the kerb. The measured response of the stringers to the known vehicle is up to 85% larger than the expected response from the grillage analysis. The causes for this difference between the theoretical and recorded results could be the influence of the guardrail, or the loss of some composite action between the steel stringers and the concrete deck.

The dynamic response of the main stringers in the structure was quite large as illustrated by the waveforms already presented in this report. 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:

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

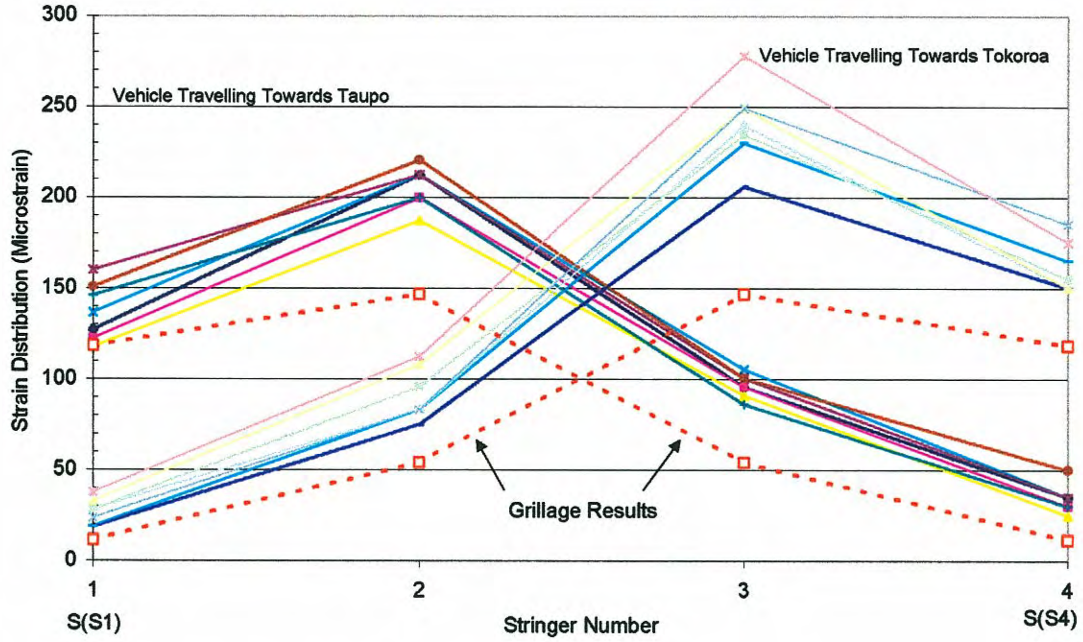


Figure 4.20 Strain distribution for known vehicle testing.

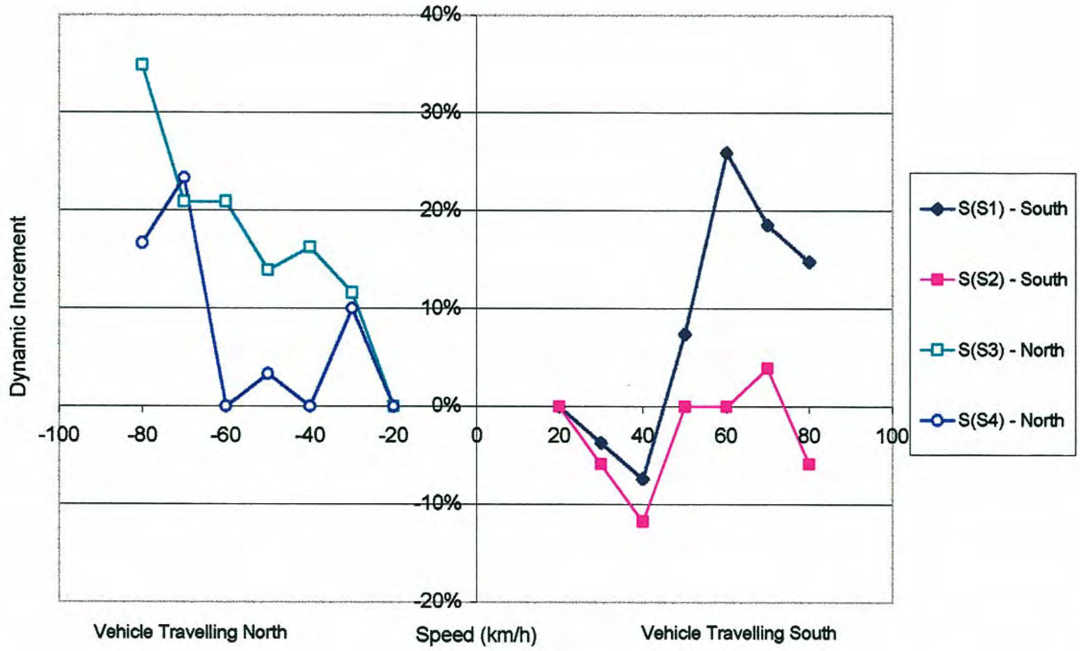


Figure 4.21 Dynamic increment plot for known vehicle travelling north to Tokoroa and south to Taupo.

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 is illustrated in Figure 4.21, and these results show a high dynamic response at 80 km/h for the known vehicle travelling north towards Tokoroa. Only the two transducers most effected by the passage of the vehicle in each direction are presented in Figure 4.21. The maximum value of 35% was recorded at 80 km/h, a value which should be adopted for the dynamic increment (impact factor) for this structure.

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. The results for the Health Monitoring were higher than the maximum recorded midspan strains for the known vehicle.

Table 4.3 Summary of health monitoring data and known vehicle test results.

Transducer	Maximum Recorded Value (Known Vehicle)	Maximum Recorded Value (Health Monitoring)	Extrapolated Value (95% confidence limit) for 100 years
	Strain ($\mu\varepsilon$)		
S(S1)	160	190	270
S(S2)	220	335	450
S(S3)	280	445	580
S(S4)	185	255	390
CG(Abut)	120	285	450
CG(1)	120	250	350
F(BC1)	80	110	165
F(BC2)	80	120	180
Slip(3)	–	1565	2000

Figure 4.22 illustrates the difference in the magnitude of the known vehicle response to the health monitoring data, which show results up to 60% higher than the known vehicle results. This difference may be due to the overloading of the structure by heavy vehicles travelling in the two directions.

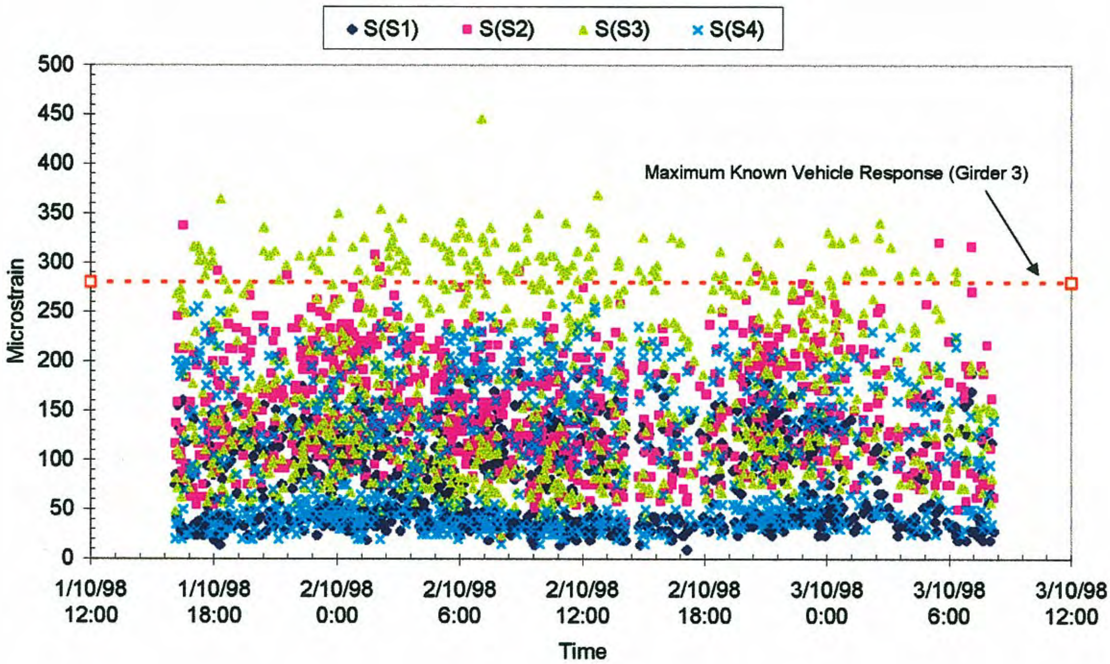


Figure 4.22 Comparison between maximum recorded values for known vehicle and the health monitoring data.

5. Fitness for Purpose Evaluation

The structural analysis described in section 3.2 of this report indicated that midspan bending of the stringers is the critical mode of failure for the structure. However since the cross girders had little reserve capacity, it was appropriate to also evaluate their capacity. This section presents the Fitness for Purpose Evaluation of the superstructure.

5.1 Multiple Presence

The Atiamuri 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 at the time of monitoring 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 this Atiamuri Bridge, and part of this project.

5.2 Stringer Bending Fitness for Purpose Evaluation

The analysis in section 3.2 of this report indicated that midspan bending was the critical mode of failure for the stringers, and the Fitness for Purpose has been determined based on this. Based on the findings in section 4, the loss of composite action is apparent. Therefore, by conservatively assuming that there is zero composite action, the strength of the section is based on the steel stringer only. The yield stress of the steel is 230 MPa (as specified by the Bridge Manual) giving a yield strain (0.85 ε_u) equal to 980 $\mu\varepsilon$.

Figure 5.1 summarises an assessment of the multiple presence effects for midspan bending strain. The diagram is based on the health monitoring data using a method that is consistent with that used in the Bridge Manual. The diagram shows a transverse distribution of strain for each direction and the sum of these two distributions. This has been completed for the 95% in 100 year event.

The data show that the highest strain due to a multiple presence event occurs in Stringer 3, and that it was equal to 925$\mu\varepsilon$ (95% in 100 years).

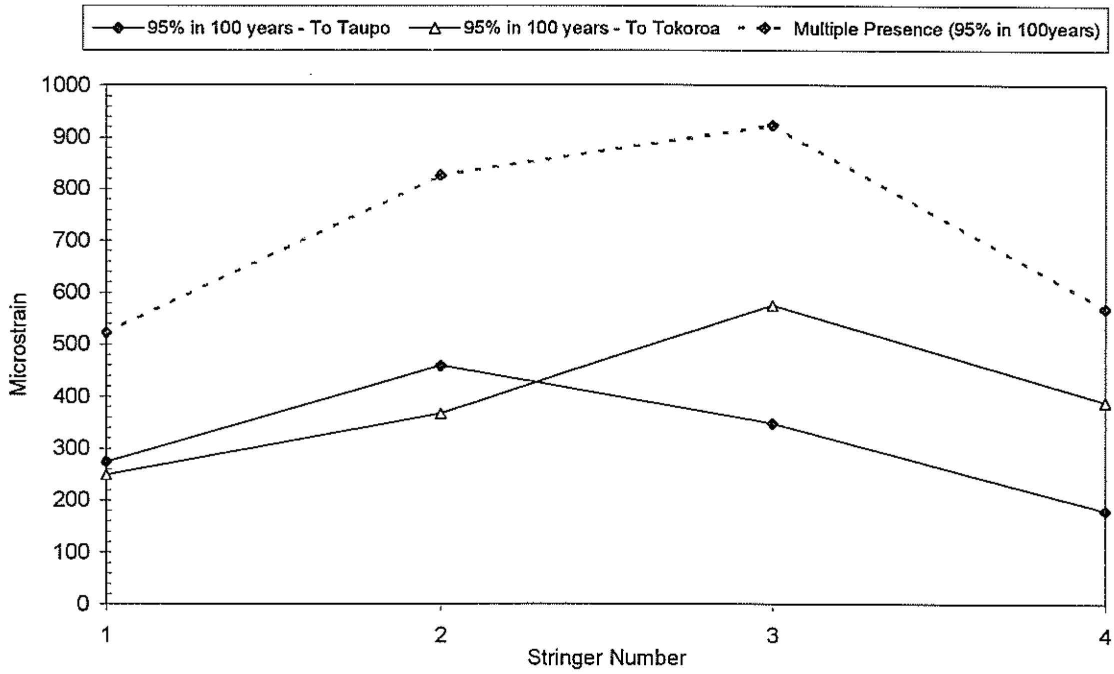


Figure 5.1 Multiple presence effects using the approach that is consistent with the Bridge Manual.

Table 5.1 Summary of Fitness for Purpose Evaluation based on the Ultimate Bending Capacity of Stringer 3.

Item	Result
Yield Strength ($\phi\varepsilon_u$)	980 $\mu\varepsilon$
Dead Load (*1.3)	250 $\mu\varepsilon$
Ultimate Live Load Capacity – Yield ($\gamma_o R_o$)	730 $\mu\varepsilon$
Ultimate Traffic Load Effect (Multiple Presence)	925 $\mu\varepsilon$
Fitness for Purpose Evaluation	80%

The Fitness for Purpose Evaluation based on the ultimate bending strength of the central stringer is presented in Table 5.1, which also summarises the calculation of the evaluation. The method for the calculation of this evaluation is outlined in section 2 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 based on the yield strength of a typical stringer for this bridge is 80%. This evaluation compares well with the 0.85 HO + 0.85 HN rating evaluation (90%) and the 2x 0.85 HN posting evaluation (105%). The comparison with the 0.85 HN loading is the most appropriate as this evaluation is related to ambient heavy vehicle traffic.

5.3 Cross Girder Bending Fitness for Purpose Evaluation

The analysis in section 3.2 of this report showed that, while cross girder bending capacity was not critical, there was limited reserve capacity above the controlling stringer capacity. Therefore the Fitness for Purpose of the cross girders has been determined. The approach is the same as that used in section 5.2. Again, the yield stress of the steel is 230 MPa (as specified by the Bridge Manual) giving a yield strain (0.85 ε_u) equal to 980 $\mu\varepsilon$.

The Fitness for Purpose Evaluation based on the ultimate bending strength of the abutment cross girder is presented in Table 5.2. The method for the calculation of this evaluation is outlined in section 2 of this report. The Fitness for Purpose Evaluation, based on the yield strength of the abutment cross girder for this bridge, is 106%. This evaluation compares well with the 0.85 HO rating evaluation (95%) and the 2x 0.85 HN posting evaluation (125%). The comparison with the 0.85 HN loading is the most appropriate to apply as this evaluation is related to ambient heavy vehicle traffic.

Table 5.2 Summary of Fitness for Purpose Evaluation based on the Ultimate Bending Capacity of the abutment cross girder.

Item	Result
Yield Strength ($\phi\varepsilon_u$)	980 $\mu\varepsilon$
Dead Load (*1.3)	21 $\mu\varepsilon$
Ultimate Live Load Capacity Strain – Yield ($\gamma_o R_o$)	959 $\mu\varepsilon$
Ultimate Traffic Load Effect (Multiple Presence)	900 $\mu\varepsilon$
Fitness for Purpose Evaluation	106%

Cross girder 1 has a larger area contributing to the load than the abutment cross girder. This larger area would normally result in greater load and, given the similarity between the cross girders, a lower Fitness for Purpose Evaluation could be expected for cross girder 1. A Fitness for Purpose Evaluation, calculated as in Table 5.2, was completed for this cross girder, and was 135%. Results from the known vehicle testing showed that the peak strain was the same in both the end and first internal cross girders (120 $\mu\varepsilon$, Table 4.3).

However, dynamic effects associated with the road profile at the southern end of the bridge are possibly causing increased strain in the end cross girder, hence giving a lower Fitness for Purpose Evaluation for the abutment cross girder.

5.4 Summary

The Fitness for Purpose Evaluation for the Atiamuri Bridge is based on the bending of the main stringers and is equal to 80%. This is considerably different to the posting evaluation based on the 2x 0.85HN vehicle load of 105% calculated by Infratech. The reasons for the large difference between the Fitness for Purpose Evaluation and the conventional load rating (from the TNZ Structural Inventory) are as follows:

- The ambient heavy vehicle traffic is inducing responses in the bridge up to 60% larger than the known vehicle and 35% larger than the 0.85 HN vehicle. Vehicle mass or dynamic effects could contribute to this result.
- A loss of composite action between the steel stringers and the concrete deck is causing higher strains and a reduction in the design strength of the section.
- The road profile may be causing amplified dynamic effects in the stringers of the monitored span. The general dynamics of the structure are also very lively, resulting in amplified strains.
- The assumed concrete strength and Young's Modulus values for this structure are low. These values have a significant effect on the resulting strength of the structure, particularly for shear.

The Fitness for Purpose of the cross girders was also evaluated in a manner similar to that for the stringers. The abutment cross girder was more critical than the first internal cross girder, and its Fitness for Purpose Evaluation was 106%. Known vehicle testing showed that the peak cross girder strains were the same for the abutment cross girder and the first internal cross girder. However the Fitness for Purpose Evaluation for the first internal cross girder was 135%. This compares reasonably well to the posting evaluation based on the 2x 0.85HN vehicle load of 125%, as calculated by Infratech.

6. Conclusions

This report presents the details and results of both a conventional rating, and a Health Monitoring-based Fitness for Purpose Evaluation for the Atiamuri Bridge.

Efficient Health Monitoring requires placement of instrumentation in the most appropriate locations (i.e. those having greatest influence on bridge capacity or performance). Structural analysis of the bridge was used to identify these locations before installing the instrumentation on site. Midspan bending of the main girders and cross girders were the governing factors affecting the capacity of the bridge. In addition, the extent of composite actions between the stringers and deck was also identified as an important parameter in determining bridge capacity.

Theoretical Analysis

The TNZ Structural Inventory (1999) has a rating (Class) for the bridge of 104% and a Deck Capacity Factor (DCF) of 1.02.

The theoretical assessment of the superstructure of the bridge made by Infratech gave the rating evaluation (0.85 HO + 0.85HN loading) as 90% and governed by stringer bending capacity. This assumed partial shear connection between the stringers and the deck in accordance with the Bridge Manual (TNZ 1994).

The cross girders were also assessed, and their rating evaluation was 95%, governed by midspan bending of the cross girder when the known 0.85 HO evaluation vehicle travels along the centre of the bridge.

Health Monitoring Results

The Health Monitoring results for Atiamuri Bridge are summarised as follows:

- The central stringers are subject to greater stress than the edge stringers. This is to be expected to some extent, although monitoring indicates that the kerbs (and possibly the guardrails as well) provide additional stiffness and strength in the vicinity of the edge stringers.
- Full composite action is not occurring between the deck and the stringers, because significant slip was recorded between these components on Stringer 3. This has a significant effect on stringer capacity. While the slip was not fully quantified in this investigation, it warrants further investigation.
- Strains recorded in Stringer 3 were significantly greater than those in Stringer 2 for equivalent known events. The cause of this difference in behaviour is the slip (and subsequent loss of composite action) noted above.

- The ambient heavy traffic produced structural responses that were significantly greater (up to 60%) than the response produced by a known heavy vehicle operating at legal load levels. This effect may be the result of overloaded heavy vehicles on the route, or could also result from dynamic effects caused by the ambient traffic, or multiple presence effects.
- The response of the abutment cross girders to ambient traffic is greater than expected. This may be the result of increased dynamic activity at the bridge abutment interface.
- The natural frequency of the bridge was difficult to determine, but one of the dominant frequencies recorded was 15 Hz. The bridge has a general dynamic response greater than would normally be expected, and a number of frequencies appear to contribute to this response.
- The maximum dynamic increment recorded for the bridge was 35%, which is reasonably consistent with the value of 1.3 recommended by the Bridge Manual (TNZ 1994) for the impact factor.

Fitness for Purpose Evaluation

The Fitness for Purpose Evaluation for this bridge, based on midspan bending of the stringers, was 80%. The theoretical posting evaluation for the bridge is 105%.

This difference suggests that the bridge is not performing as well as might be expected based on theoretical calculations. This poor performance could be the result of deterioration in the composite action between the deck and the stringers.

The theoretical rating showed that cross girders had only slightly more reserve capacity than the stringers. The Fitness for Purpose Evaluation of the cross girders, based on Health Monitoring, was 106%. This suggests that stringer performance is the critical issue with respect to bridge deck capacity.

7. Recommendations

Infratech is currently completing a long-term Health Monitoring programme for the Atiamuri Bridge. Once the long-term results are available, the results of both investigations will be compared.

Based on the conclusions in section 6, the following recommendations are made:

- Undertake further investigations into the extent of composite action mobilisation on the Atiamuri Bridge.

  The behaviour of the deck–stringer shear connection may have deteriorated to a greater extent that that predicted by the Bridge Manual (TNZ 1994). If so, this can result in a significant reduction in bridge capacity, and should be monitored.

- Monitor the composite action, and the associated deterioration, of the Atiamuri Bridge, to gain a better understanding of the phenomenon, because it will be relevant to other New Zealand bridges.

A correctly designed Health Monitoring programme could achieve the following objectives:

- Manage the risk of failure at the Atiamuri Bridge by continuous monitoring.
- Obtain an improved understanding of this deterioration phenomenon that can be used to manage the New Zealand bridge population.
- Determine the most appropriate rehabilitation strategy to ensure that maximum service life is obtained from the existing bridge.

8. References

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