Health Monitoring of Superstructures of New Zealand Road Bridges:

Wanganui Bridge, Wanganui

Transfund New Zealand Research Report No.172

Health Monitoring of Superstructures of New Zealand Road Bridges:

Wanganui Bridge, Wanganui

R.P. Andersen, W.S. Roberts, R.J. Heywood & T.J. Heldt
Infratech Systems & Services Pty Ltd,
Brisbane, Australia

Transfund New Zealand Research Report No. 172

ISBN 0-478-11580-0
ISSN 1174-0574

PO Box 2331, Lambton Quay, Wellington, New Zealand
Telephone 64-4-473 0220; Facsimile 64-4-499 0733

Andersen, R.P., Roberts, W.S., Heywood, R.J., Heldt, T.J. 2000. Health monitoring of superstructures of New Zealand road bridges: Wanganui Bridge, Wanganui. *Transfund New Zealand Research Report No. 172.* 44pp.

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

An Important Note for the Reader

The research detailed in this report was commissioned by Transfund New Zealand. Transfund New Zealand is a Crown entity established under the Transit New Zealand Act 1989. Its principal objective is to allocate resources to achieve a safe and efficient roading system. Each year, Transfund New Zealand invests a portion of its funds on research that contributes to this objective.

While this report is believed to be correct at the time of its preparation, Transfund New Zealand, and its employees and agents involved in the preparation and publication, cannot accept any liability for its contents or for any consequences arising from its use. People using the contents of the document, whether direct or indirect, should apply, and rely upon, their own skill and judgement. They should not rely on its contents in isolation from other sources of advice and information. If necessary they should seek their own legal or other expert advice in relation to their circumstances and the use of this report.

The material contained in this report is the output of research and should not be construed in any way as policy adopted by Transfund New Zealand but may form the basis of future policy.

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 Renall Haulage, Masterton, 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.

CONTENTS

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 Wanganui Bridge, on State Highway 4, crosses the Wanganui River near Taumarunui, Wanganui Region, North Island. It was selected as one of these ten because of its relatively low conventional strength evaluation and the spans are relatively long (30.5 m). This 140 m-long bridge was built in 1962, is a significant transport infrastructure asset, and is representative of a large population of prestressed concrete bridges in New Zealand.

This report focuses on the 30.5 m spans of this bridge. These spans consist of five segmental post-tensioned T beams, consisting of 11 precast concrete T beam segments, longitudinally post-tensioned together.

Theoretical Analysis

The results of the evaluations based on flexural capacity are greater than those for web shear. Consequently, the web shear capacity of the main girders is the critical issue and determines the capacity of the superstructure. The theoretical rating evaluation based on the web shear is 45% and the posting evaluation is 55%, assuming that 21 MPa concrete was used. The rating evaluation is 65% assuming use of 40 MPa concrete. The rating evaluation from the 1999 Transit New Zealand Structural Inventory is 61%. It is understood that, according to normal practice, this bridge would require posting.

The serviceability limit state (with no tensile stress across construction joints) also limits the bending capacity of the structure. The theoretical rating and posting evaluations based on this criterion are 85% and 110% respectively. The performance of the deck was not considered in this report.

Health Monitoring Results

The results of the Health Monitoring programme show that:

- The ambient heavy vehicle traffic is inducing bending moments in the bridge that are similar to those induced by the 0.85HN vehicle, and very little overloading occurs on this route.

- The recorded strains are significantly lower than the strains predicted in the analysis. This may be related to bearing restraint or to differences in actual and assumed material properties.
- The load distribution across the five girders is more uniform for the actual traffic effects compared to the distribution assumed for the rating and posting evaluation loads. This contributes to higher evaluations based on the health monitoring data.

Fitness for Purpose Evaluation

- The Fitness for Purpose Evaluation based on web shear ranged from 85% to 100% for concrete strengths of 21 MPa and 40 MPa respectively.
- For the serviceability limit state which limits the tensile stresses across the interfaces between the segments, the Fitness for Purpose Evaluation would be 140%.
- These results indicate that web shear is an issue with this bridge, and therefore a number of assumptions were made in the analysis for this report.

Recommendations

The recommendations are that:

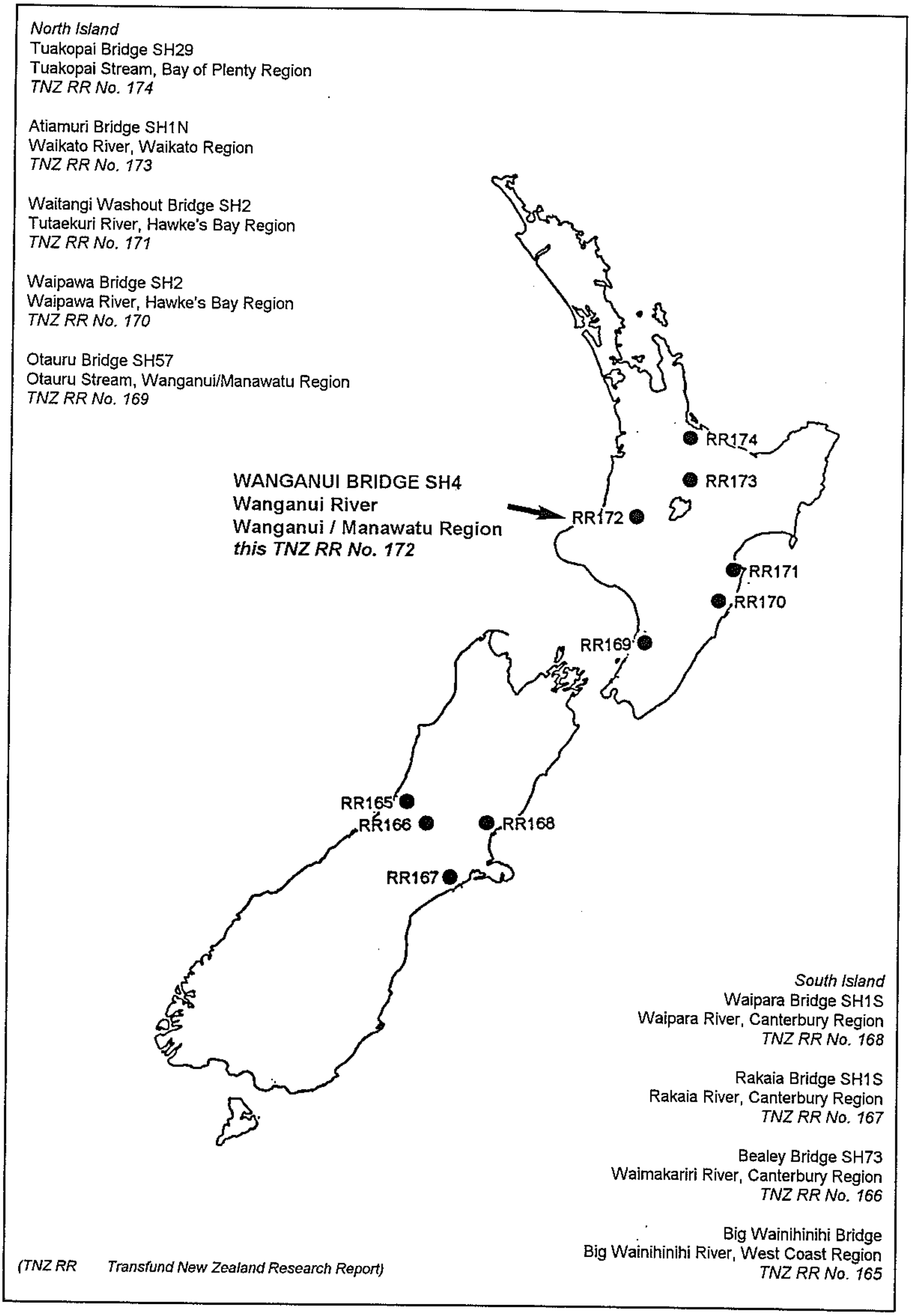
- Web shear effects should be measured more accurately using three strain gauges (rosettes) at a number of locations.
- Shear results between the individual girders should be compared to determine the critical girder that is in shear.
- The actual concrete strength should be measured, because the web shear strength is sensitive to the concrete strength, and in particular to the principal tensile strength.
- Further research into the web shear capacity of prestressed concrete bridge girders is warranted.
- The results from the Health Monitoring programme showed that the girders of this bridge may have enough shear capacity to avoid the requirement to post the bridge, but further investigations are needed.

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 Wanganui Bridge on State Highway 4, crosses the Wanganui River near Taumarunui, Wanganui Region, North Island. It was selected as one of these ten, as it has low conventional strength evaluation, and the spans are relatively long (30.5 m). It is a significant transport infrastructure asset, and is representative of a large population of prestressed concrete bridges in New Zealand.

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



1. Introduction

1.1 Background

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

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 Wanganui Bridge, on State Highway (SH) 4, which crosses the Wanganui River near Taumarunui, Wanganui Region, North Island of New Zealand (Figure 1.1).

The reasons for choosing this bridge for the representative sample were:

- It has a low conventional strength rating.
- It represents a significant asset in New Zealand's transport infrastructure.
- It has relatively long spans (30.5 m).
- It is representative of a significant population of prestressed concrete bridges.

The objective of this investigation was to evaluate the Fitness for Purpose of the superstructure (30.5 m spans) of the Wanganui 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.

This evaluation is based principally on the following components of the Wanganui Bridge structure:

- Midspan bending strength of the main prestressed concrete girders.
- Shear strength of the main prestressed concrete girders.

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.

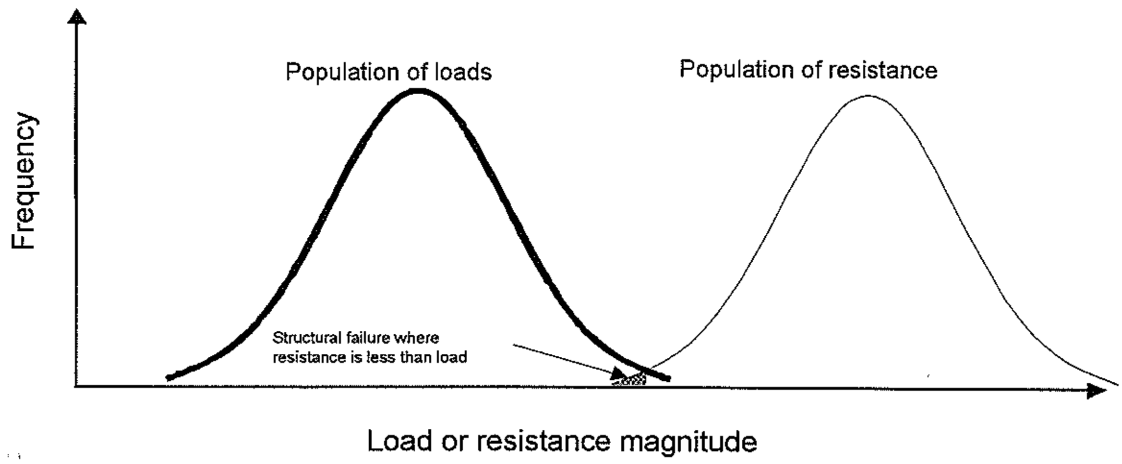


Figure 2.1 Statistical representation of structural failure.

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

2.3.1.1 *Rating Evaluations*

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

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

2.3.1.2 *Posting Evaluations*

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

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

2.3.2 Decks

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

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

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

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

2.4 The Health Monitoring Approach

2.4.1 Theory of this Approach

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

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

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

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

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

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

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

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

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

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

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

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 Wanganui 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 Wanganui 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 Wanganui Bridge located on State Highway (SH) 4, crosses the Wanganui River near Taumarunui. It consists of six simply supported spans, including three spans of 15.2 m and 3 spans of 30.5 m. The 15.2 m spans consist of five segmental post-tensioned concrete T beams with a reinforced concrete deck.

The 30.5 m spans, which are the subject of this report, consist of five segmental post-tensioned T beams. These T beams consist of 11 precast concrete T beam segments which are longitudinally post-tensioned together. An in-situ jointing concrete is used between the segments. The top flange of the T beams forms the deck with a reinforced concrete infill in the longitudinal joints between the flanges. The bridge also has transverse post-tensioned diaphragms.

Construction of the 140 m-long bridge was completed in 1962. The bridge is illustrated in Figure 3.1, and a typical cross section of the 30.5 m spans is illustrated in Figure 3.2.

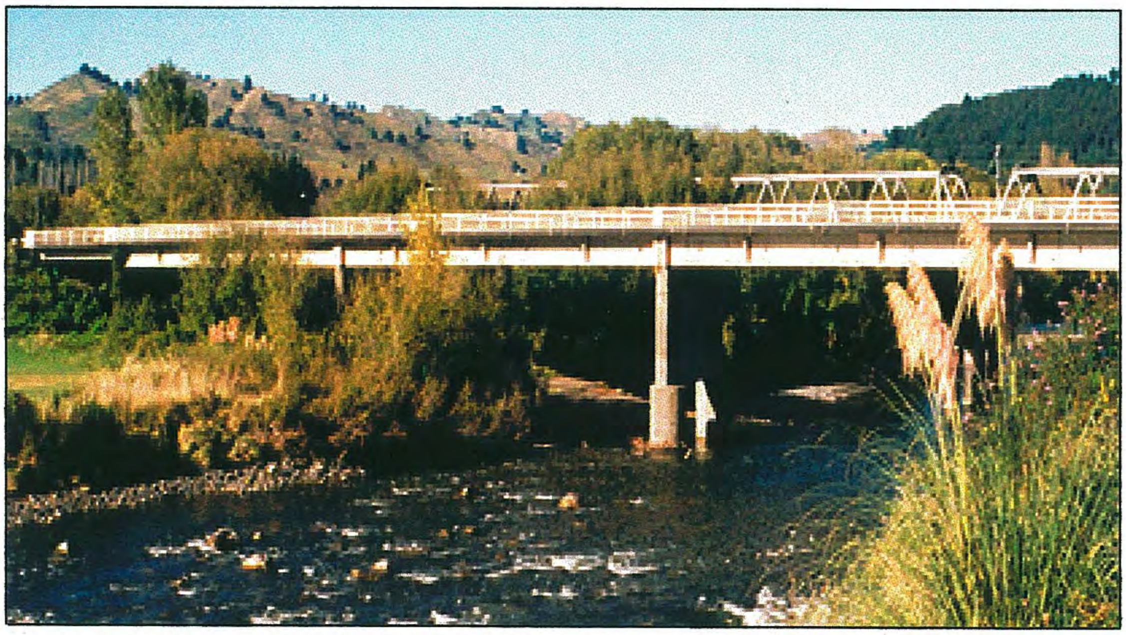


Figure 3.1 Wanganui Bridge, central North Island, New Zealand.

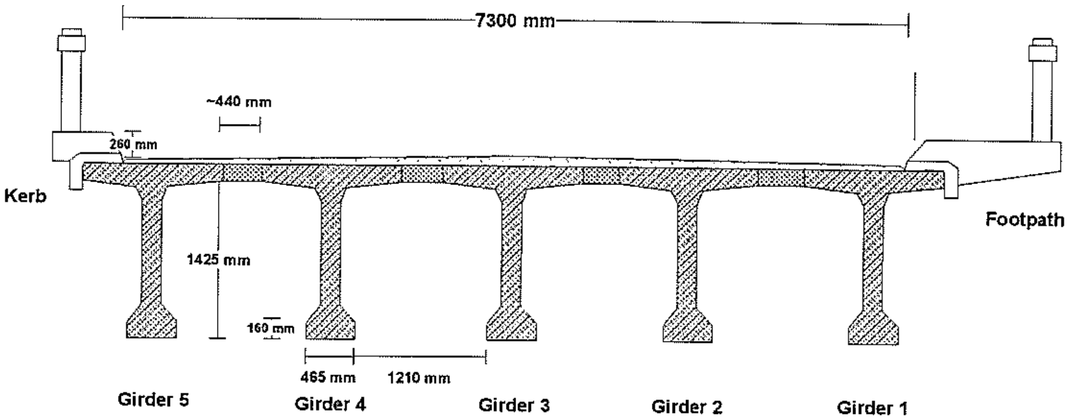


Figure 3.2 Typical cross section of the 30.5 m spans.

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

- Bridge Classification (superstructure) 61%
- Deck Capacity Factor (DCF) 1.0

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, and were confirmed by on-site measurements.

The material properties for the concrete were not available. Instead the properties used were obtained from Section 6.3.4 of the Bridge Manual. The material properties used in the analysis of this bridge are listed below, where f_p is the tensile strength of the prestressing strands (other nomenclature as in the Bridge Manual):

- Concrete Girders and Deck f'_c = 21 MPa, E = 22 100 MPa
- Reinforcement f_y = 250 MPa, E = 200 000 MPa
- Prestressing Strands f_p = 1700 MPa, E = 200 000MPa

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

3.2.1 Girder Bending

The maximum bending moment in the girders resulting from the dead load is 2235 kNm/girder. A summary of the maximum bending moments in a typical 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 represent the maximum bending moment in a girder with the vehicles at the greatest allowable eccentricity.

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

Load	Bending Moment (kNm)
Dead Load	2235
Known Vehicle	650
2x 0.85HN Vehicles (Posting Load)	1105
0.85HO + 0.85HN Vehicles (Rating Load)	1500

The bending capacity of the concrete girders in the superstructure, calculated in accordance with Section 16 of the Concrete Structures Standard (NZS 3101: Part 1 1995), and the ultimate moment capacity (ϕM) is 7080 kNm.

The Concrete Structures Standard also imposes a limit of no tensile stress in the construction joints between the segments of the girders. This is a serviceability requirement and also has the potential to limit the loads on this structure. The maximum moment that can be applied to the concrete girders based on this limit is 3740 kNm.

3.2.2 Girder Shear

The maximum shear forces obtained from the grillage analysis are presented in Table 3.2. The edge girders are critical with respect to web shear, and the shear forces presented in this table are for a typical edge girder.

The shear capacity of the main girders was evaluated in accordance with Section 9 of the Concrete Structures Standard (NZS 3101: Part 1 1995). The girders have the potential to fail in either flexure shear or web shear modes. The flexural shear capacity (ϕV) is 705 kN, and the web shear capacity (ϕV) is 557 kN. The critical failure mode for shear in these girders is web shear.

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

Load	Shear Force (kN)
Dead Load	290
Known Vehicle	100
2x 0.85HN Vehicles (Posting Load)	140
0.85HO + 0.85HN Vehicles (Rating Load)	215

3.2.3 Deck Capacity

The capacity of the deck was not evaluated in this investigation. The Deck Capacity Factor (DCF) for this bridge based on the TNZ Structural Inventory is 1.0, which indicates that the deck has adequate capacity to resist the applied traffic loads.

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 evaluation of the structure are presented in Table 3.3, with posting and rating evaluations assessed for the bending and shear in the girders. The table also presents a comparison of the evaluation calculated by Infratech Systems & Services (Infratech), and the posting and rating evaluations recorded in the current (1999) TNZ Structural Inventory.

A value of 1.22 for the impact factor and a value of 1.3 for the dead load factor was used in the load evaluations.

The rating and posting loads presented in the table do not include impact factors, but these are included in the rating and posting evaluations (percentages).

The results in this table show that the serviceability limit state governs the bending strength, but the serviceability limit state for bending is also sensitive to prestress losses. However it is the web shear capacity that governs the capacity of the superstructure. The rating evaluation based on this failure mode is 45% and the posting evaluation is 55%.

The web shear capacity is sensitive to the concrete strength, and in particular to the principal tensile strength. The web shear capacity in this investigation was calculated assuming a concrete compressive strength of 21 MPa. Increasing this strength to 40 MPa (which is a reasonable assumption) increases the rating evaluation from 45% to around 65%. The value of 21 MPa is a lower bound suggested in the Bridge Manual, but the drawings indicate that the girders were to have 34 MPa strength at 28 days. The rating evaluation from the TNZ Structural Inventory is 61%.

As the posting evaluation of this structure is less than 100%, the normal practice would be to post the bridge, but it is understood that the bridge is currently not posted.

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

<table>
<tr><th>Mode of Failure</th><th>φ Ultimate Capacity</th><th>0.85 HO Rating Load</th><th>0.85 HN Posting Load</th><th>Dead Load</th><th>0.85 HO Rating (Infratech)</th><th>0.85 HN Posting (Infratech)</th><th>Rating (Structural Inventory)</th></tr>
<tr><td>Girder Bending (Ultimate)</td><td>7080kNm</td><td>1500kNm</td><td>1105kNm</td><td>2235kNm</td><td>155%</td><td>165%</td><td rowspan="4">61%</td></tr>
<tr><td>Girder Bending (Service)</td><td>3740kNm</td><td>1500kNm</td><td>1105kNm</td><td>2235kNm</td><td>85%</td><td>110%</td></tr>
<tr><td>Flexural Shear</td><td>700kN</td><td>215kN</td><td>140kN</td><td>290kN</td><td>85%</td><td>100%</td></tr>
<tr><td>Web Shear</td><td>557kN</td><td>215kN</td><td>165kN</td><td>290kN</td><td>45%</td><td>55%</td></tr>
</table>

3.4 Summary

The Wanganui Bridge, in Wanganui Region, was analysed using a grillage analysis to determine the bending moment and shear in the girders of a typical 30.5 m span, based on various vehicle loadings. Web shear is the critical failure mode and governs the capacity of the superstructure. The serviceability limit state (with no tensile stress permitted across joints between segments) also limits the bending capacity of the main girders.

The Deck Capacity Factor is 1.0 obtained from the TNZ Structural Inventory.

Based on the results from this analysis, the Health Monitoring programme concentrated on evaluating the Fitness for Purpose of the girders based on web shear. Because the limit of no tensile stress across the joints is sensitive to prestress losses, the strains across the joints were also measured and evaluated.

4. Health Monitoring Programme

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

- Short-term health monitoring of the ambient heavy vehicle traffic for a period of approximately three 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 Wanganui Bridge.

4.1 Instrumentation

The instrumentation for this bridge included eight Demountable Strain Gauge transducers, installed on span 4 of the structure, The positions of this instrumentation are illustrated in Figure 4.1.

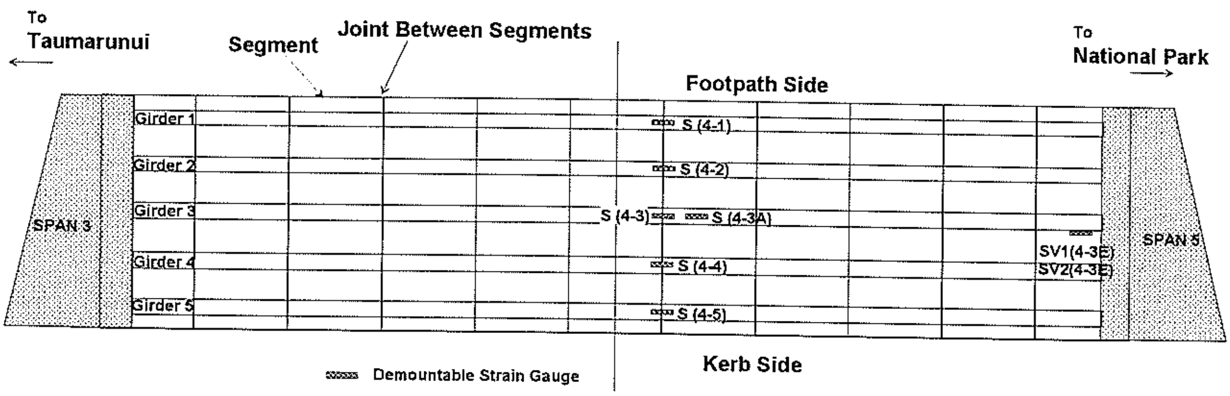


Figure 4.1 Instrumentation plan for Wanganui Bridge.

Figure 4.1 shows six transducers installed close to the midspan of the structure to record the maximum bending strains. These transducers were positioned over a joint between two of the segments making up the girders. Transducer S(4-3A) was positioned to one side of transducer S(4-3) in order the determine the difference between strains across a joint between segments, and strains in the concrete on the soffit of a segment.

The location and orientation of the transducers installed to record the web shear effects in Girder 3 are illustrated in Figure 4.2. These transducers were orientated at 45° to the horizontal axis of the girder. To accurately measure web shear effects, three strain transducers are required, positioned in a traditional rosette pattern.

For this bridge the assumption was made that the transducers were positioned at the neutral axis and thus the axial strain caused by the live load would be zero.

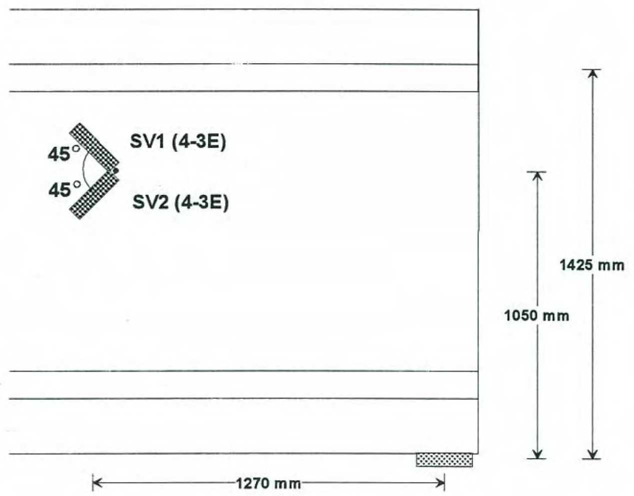


Figure 4.2 Locations of transducers for measuring web shear effects on Girder 3.

Figure 4.3 shows the installation of the demountable strain gauges on the girders at midspan.

Figure 4.3 Installation of instrumentation on girders of the Wanganui Bridge.

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

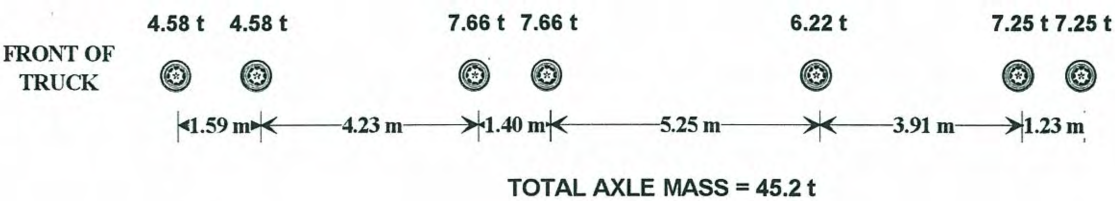
4.2 Procedure

The health monitoring of the structure began on Saturday 3 October, and continued until Tuesday 6 October, 1998, giving a total monitoring period of approximately 62 hours. During the three-day monitoring period, the response of the bridge to 144 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 dimension) was measured. This component of the Health Monitoring programme was conducted on Monday 5 October, 1998. The vehicle used for the testing, shown in Figure 4.4, was a seven-axled heavy vehicle of known mass (45.2 tonnes) and dimensions (Figure 4.5).

Figure 4.4 The known vehicle used for behavioural testing.

Figure 4.5 Axle mass and configuration of the known vehicle.



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 and 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 for each direction. 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. Figure 4.6 illustrates the vehicle on the bridge during testing, and the segments in the girders are clearly visible in this figure.

Figure 4.6 The known vehicle on the bridge during the testing.

4.3 Short-Term Health Monitoring Results

4.3.1 Girder Response

Typical response versus time graphs (waveforms) for the midspan bending strains recorded during the Health Monitoring programme, for the passage of a typical heavy vehicle are presented in Figure 4.7. Some dynamic response occurs in the bridge after the vehicle has passed over the instrumented span. The magnitude of these strains is low, resulting in waveforms that are not smooth. Under these circumstances the resolution of the monitor was too low to provide a smooth digital signal from the analogue signal (which has a small range). Higher resolution settings on the monitor would overcome this problem.

Figure 4.7 Waveform for response versus time for midspan strain transducers for event recorded at 13.18 hours, 5 October 1998.

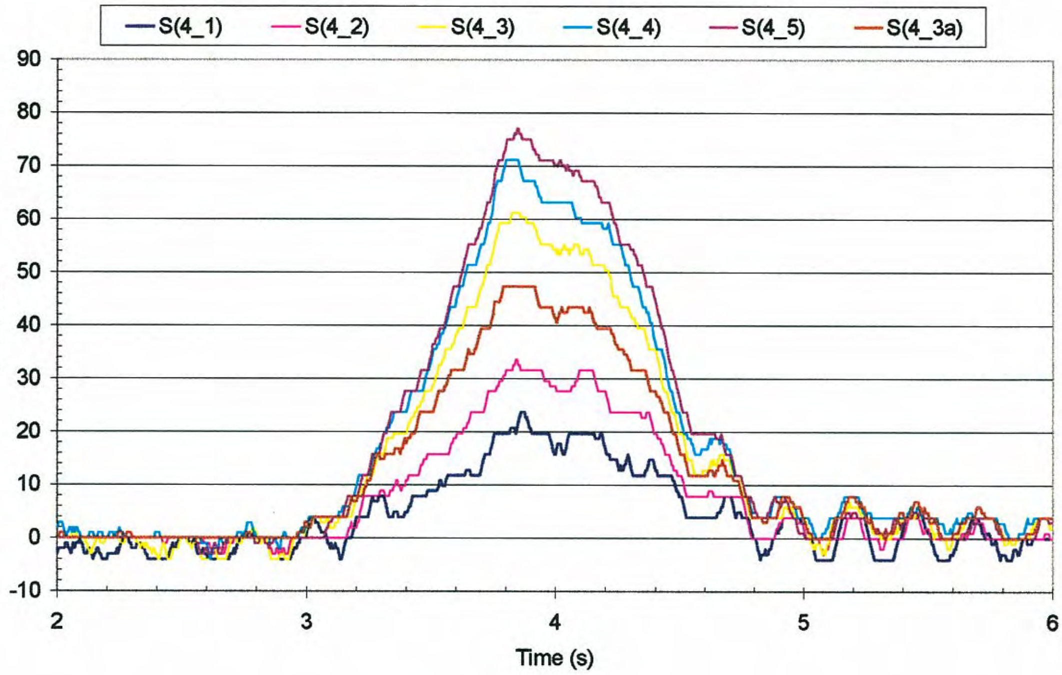
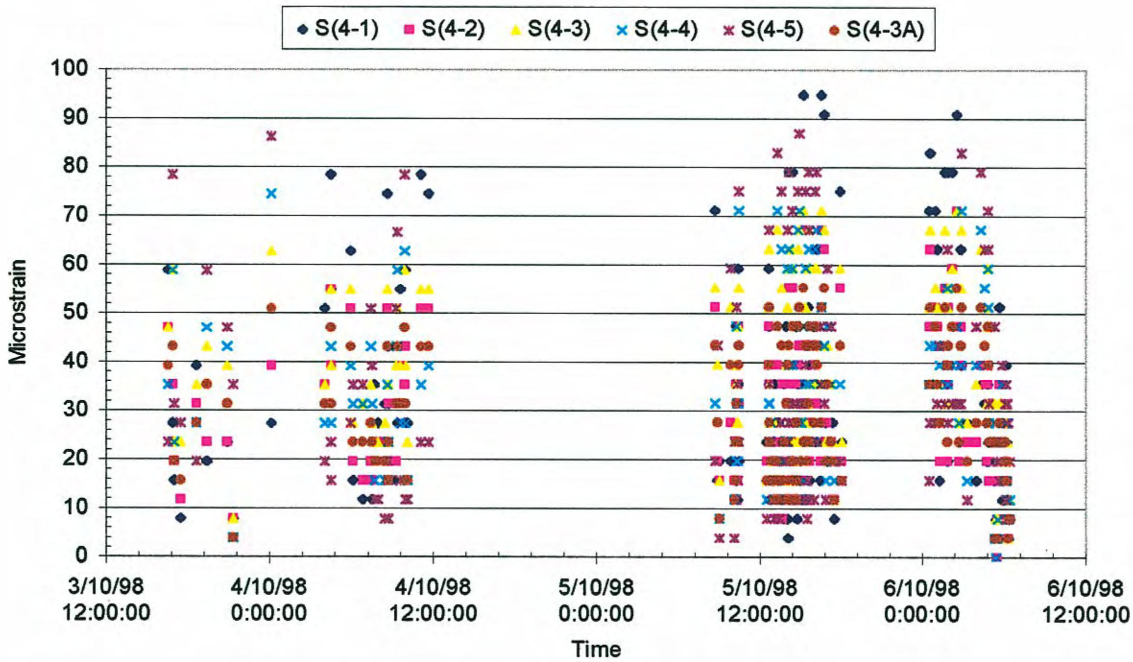


Figure 4.8 Scatter diagram for midspan bending strains.



The scatter diagram in Figure 4.8 represents the maximum bending strains recorded during the passage of each heavy vehicle for the entire Health Monitoring period.

This plot gives an indication of the characteristics of the heavy vehicles travelling over the bridge, including the distribution of mass and the number of heavy vehicles travelling this route. The large gap in the data occurred during monitor downtime, and is not due to an absence of traffic.

The scatter diagram (Figure 4.8) displays consistently higher responses from transducers S(4-1) and S(4-5) which are located on the edge girders. Figure 4.9 presents a comparison of the transducers installed approximately 500 mm apart on Girder 3. Transducer S(4-3A) is plotted on the y-axis and transducer S(4-3) is plotted on the x-axis. If a one-to-one relationship existed between the two transducers, a line with a slope of 1.0 would result. The information in this figure indicates that the strain in the soffit of the segment is approximately 75% of the strain across the joint between segments. This difference is to be expected as the segments have additional conventional reinforcement which would make the segments stiffer than the joints which have only post-tensioning.

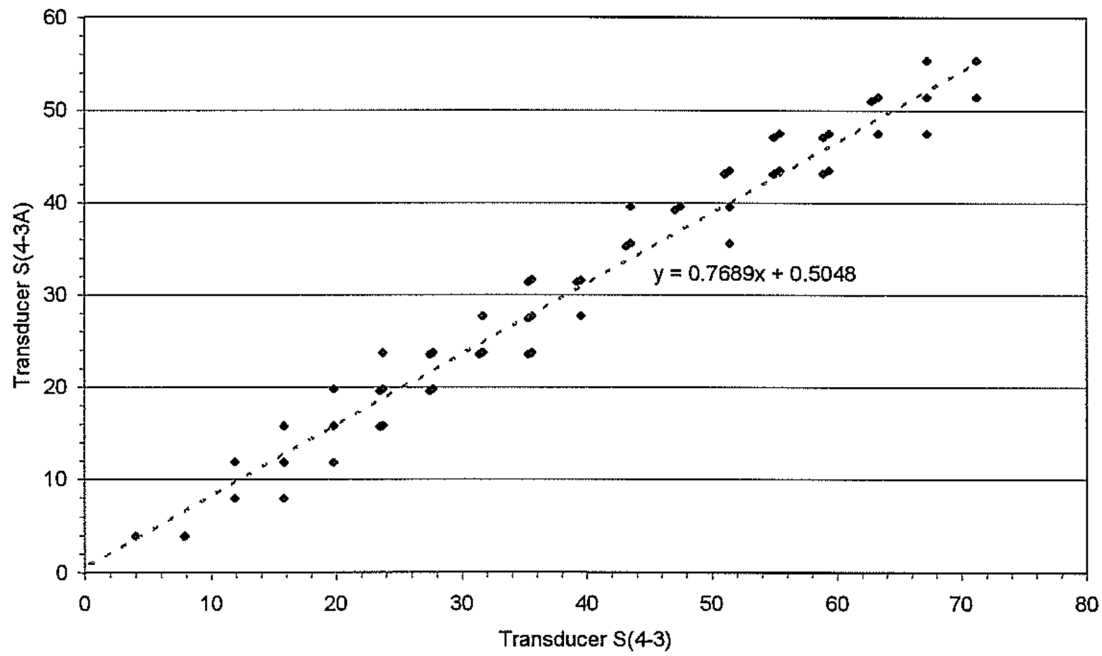


Figure 4.9 Comparison of strains across the segmental joint and in the concrete.

The response of the transducers that measured web shear effects are presented in Figure 4.10. These waveforms are not smooth because of the very small strains being recorded. Higher resolution settings on the monitor would overcome this problem. The scatter diagram for these transducers is presented in Figure 4.11, in which the data show that transducer SV1(4 -3E) measured compression, while transducer SV2(4-3E) measured tension. Note the classical shape of a shear influence line from a heavy vehicle.

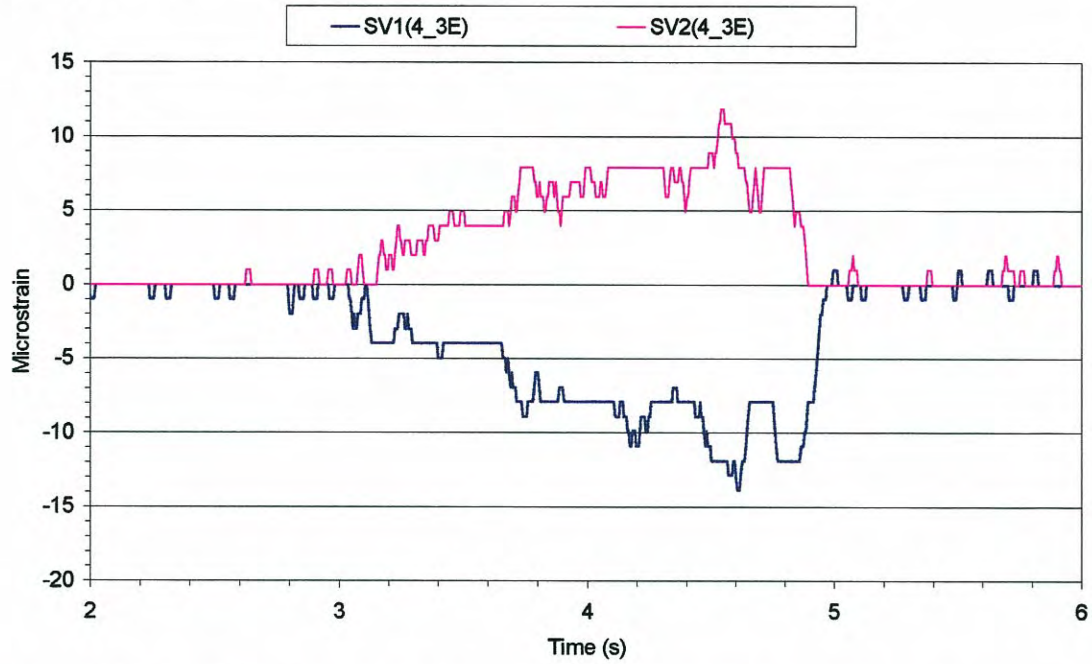


Figure 4.10 Waveform for transducers measuring web shear effects in Girder 3 for event recorded at 10.24am, 5 October 1998.

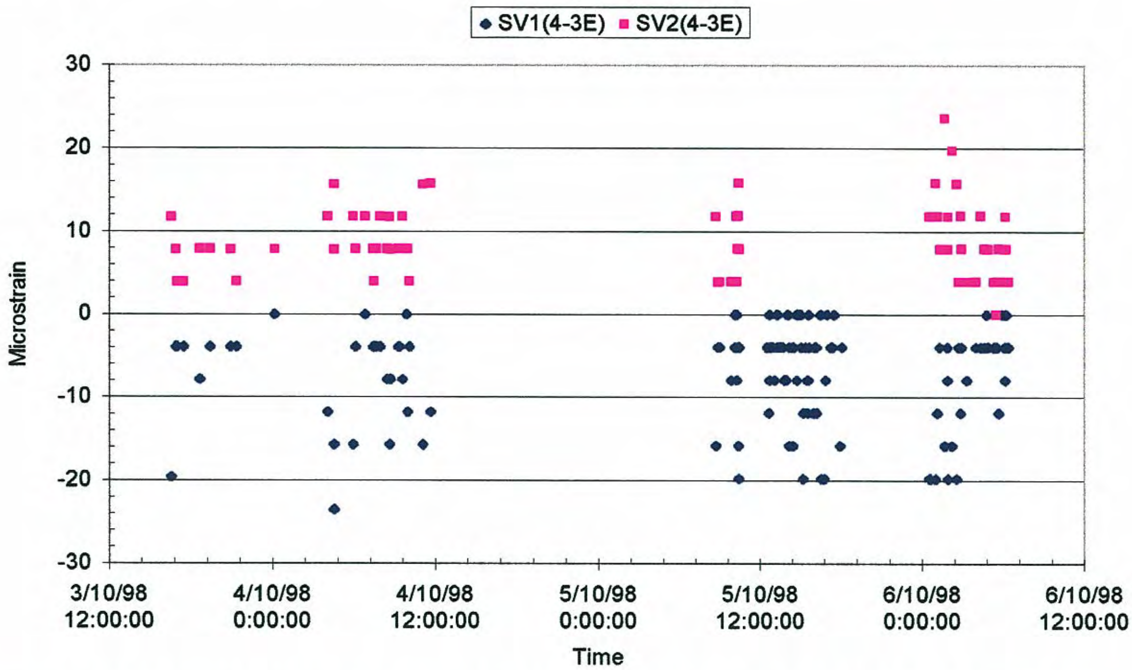
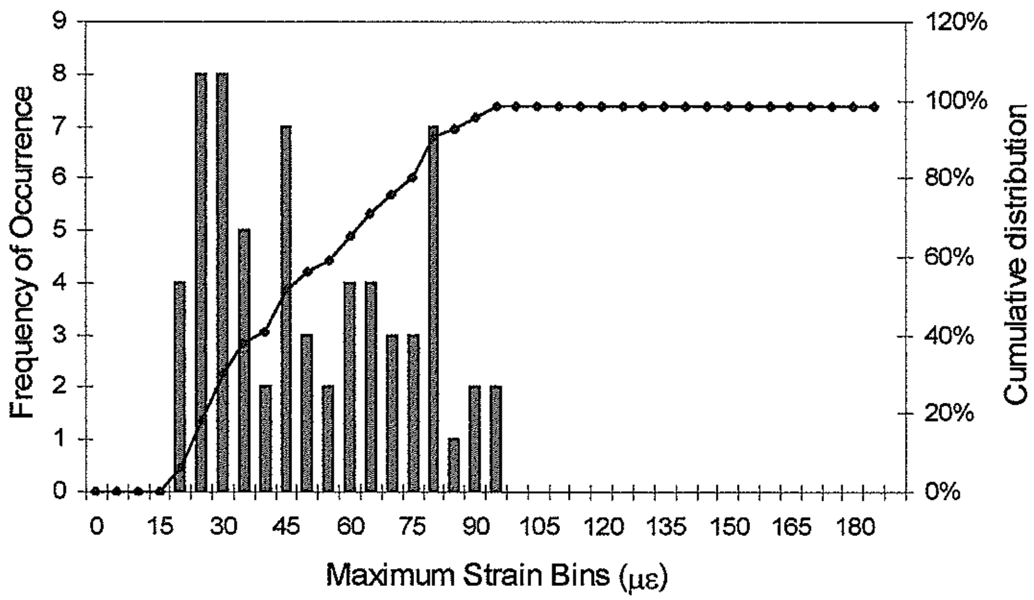


Figure 4.11 Scatter diagram for transducers measuring web shear effects in Girder 3.

4.3.2 Extrapolated Data

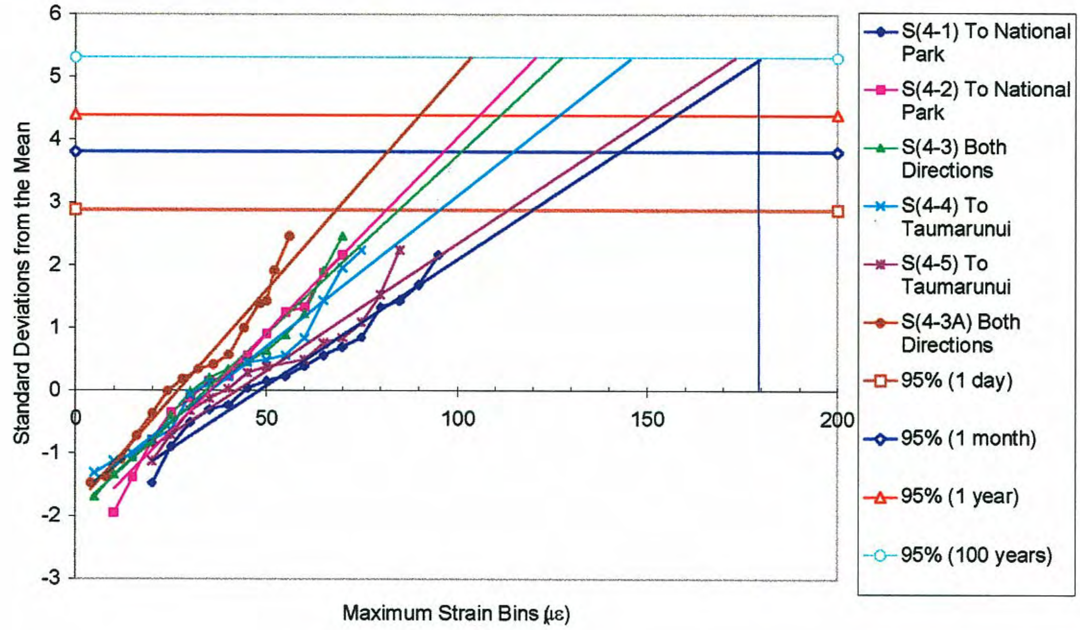
The data from the scatter diagrams can also be plotted on a histogram that incorporates a cumulative distribution. An example is presented for transducer S(4-1) in Figure 4.12. The histogram illustrates two separate sections or populations of data. This is characteristic of traffic travelling in opposite 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 indication of the traffic can be determined for each girder.

Figure 4.12 Histogram and cumulative distribution function for transducer S(4-1).



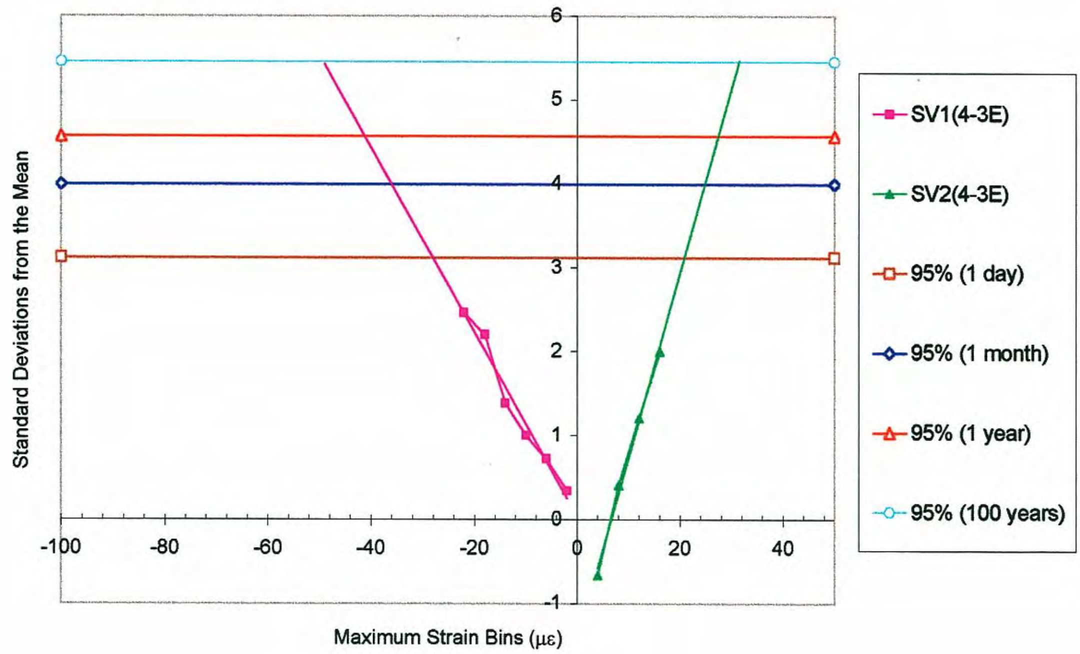
The cumulative distribution function can then be plotted on a probability scale known as an "inverse normal scale". The inverse normal plot for each of the transducers measuring midspan bending strain is presented in Figure 4.13. In this figure, the data are separated into opposite directions for all transducers, except S(4-3) and S(4-3A). These transducers are equally affected by the traffic in both directions as this girder is in the centre of the bridge. 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.

Figure 4.13 Inverse normal plot for strain transducers installed at the midspan of Span 4 girders.



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 4.14 Inverse normal plot for transducers measuring shear in Girder 3.



The inverse normal plot (Figure 4.13) 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(4-1). The extrapolated value is approximately 180 $\mu\varepsilon$. Typically the edge girders (Girders 1 and 5) are experiencing the highest strains.

The inverse normal plots for the transducers recording web shear effects installed on Girder 3 are presented in Figure 4.14.

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

Table 4.1 Extrapolated data obtained from inverse normal distribution.

Transducer	Maximum Recorded Value (Health Monitoring)	Extrapolated Value (95% Confidence limit) for 100 years
	Strain ($\mu\varepsilon$)	
S(4-1)	95	180
S(4-2)	71	125
S(4-3)	71	130
S(4-4)	75	145
S(4-5)	87	170
S(4-3A)	55	100
SV1(4-3E)	–24	–50
SV2(4-3E)	16	35

4.4 Known Vehicle Testing

A typical waveform from the testing with the known vehicle (travelling south) is presented in Figure 4.15. 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.

Table 4.2 Maximum strains recorded for known vehicle testing.

Transducer	Maximum Strains ($\mu\varepsilon$)
S(4-1)	95
S(4-2)	63
S(4-3)	67
S(4-4)	75
S(4-5)	79
S(4-3A)	51
SV1(4-3E)	–24
SV2(4-3E)	16

Figure 4.15 Typical waveform for the known vehicle travelling at 70 km/h east from Taumarunui, towards National Park.

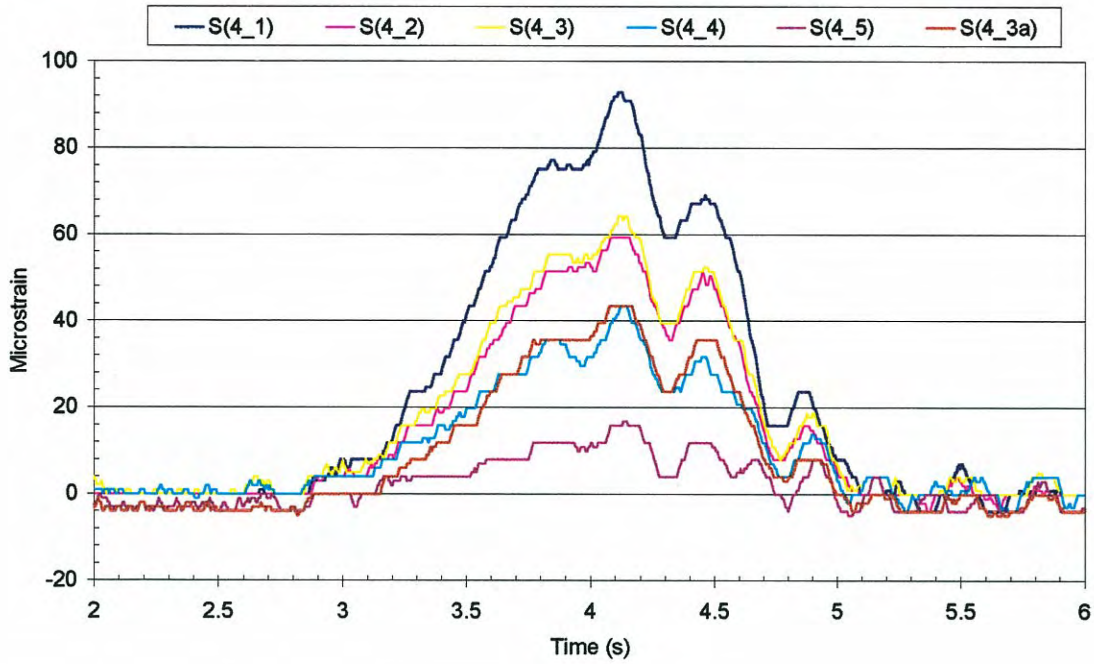
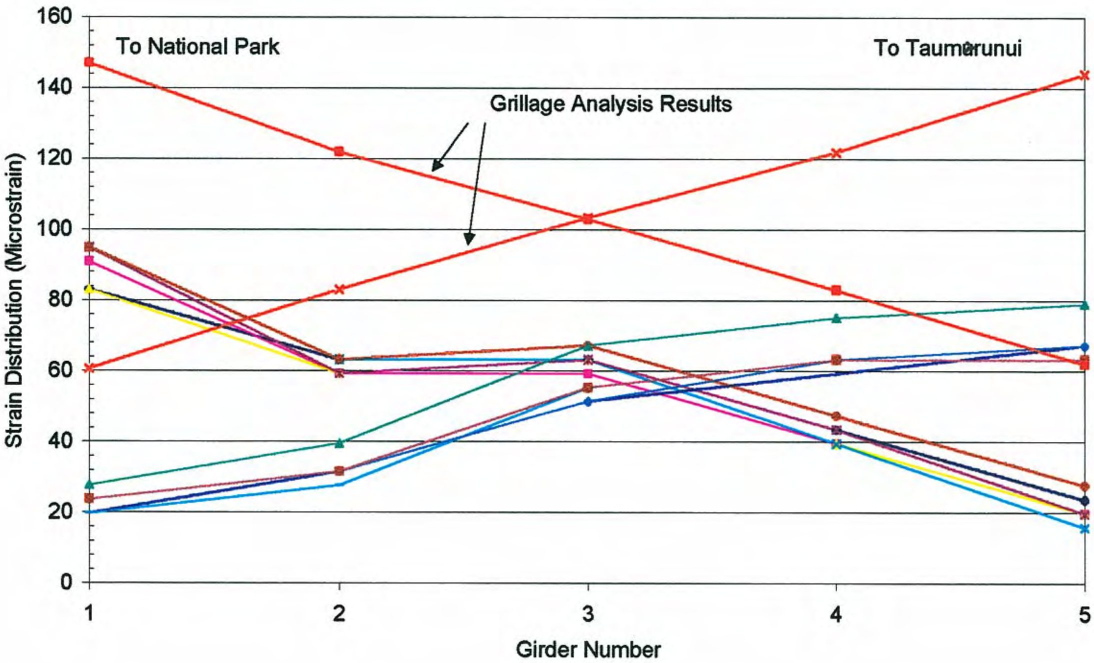


Figure 4.16 Strain distribution for known vehicle testing.



The distribution of strain into each of the girders from the known vehicle data is presented in Figure 4.16. The distribution presented is consistent with the data collected from health monitoring of the ambient heavy vehicle traffic, with higher strains recorded in Girder 1 for vehicles travelling to National Park (east). However, for vehicles travelling to Taumarunui (west), the maximum strain was recorded in Girder 5.

Figure 4.16 also illustrates the results from the grillage analysis using the known vehicle. This grillage analysis included the effects of the kerb but not the guardrail. The vehicle position for the grillage was 600 mm out from the kerb. The differences between the theoretical and recorded results indicate that the actual response of the bridge is significantly less than that predicted by the grillage analysis. These differences may be related to bearing restraint effects or to differences between actual and assigned material properties.

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

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

The response of the crawl test was used for the static result in the calculation of dynamic increment. The variation in dynamic increment for the known vehicle is illustrated in Figure 4.17. These results show a high dynamic response at 80 km/h for the known vehicle travelling west towards Taumarunui.

Only the three transducers most affected by the passage of the vehicle in each direction are presented in Figure 4.17. The maximum value of 25% was recorded at 80 km/h for the vehicle travelling towards Taumarunui. Generally the dynamic increment is below the value of 22%, calculated from the Bridge Manual, used in the theoretical analysis of the bridge in this report. The natural frequency of the span is around 4 Hz.

4.5 Summary

A summary of the data recorded for the Health Monitoring and the testing with the known vehicle is presented in Table 4.3. For the testing of the Wanganui Bridge, the maximum recorded midspan strains for the known vehicle were the same as the results for the Health Monitoring. This indicates that very little overloading occurs on this route. The highest strains were recorded in the two edge girders.

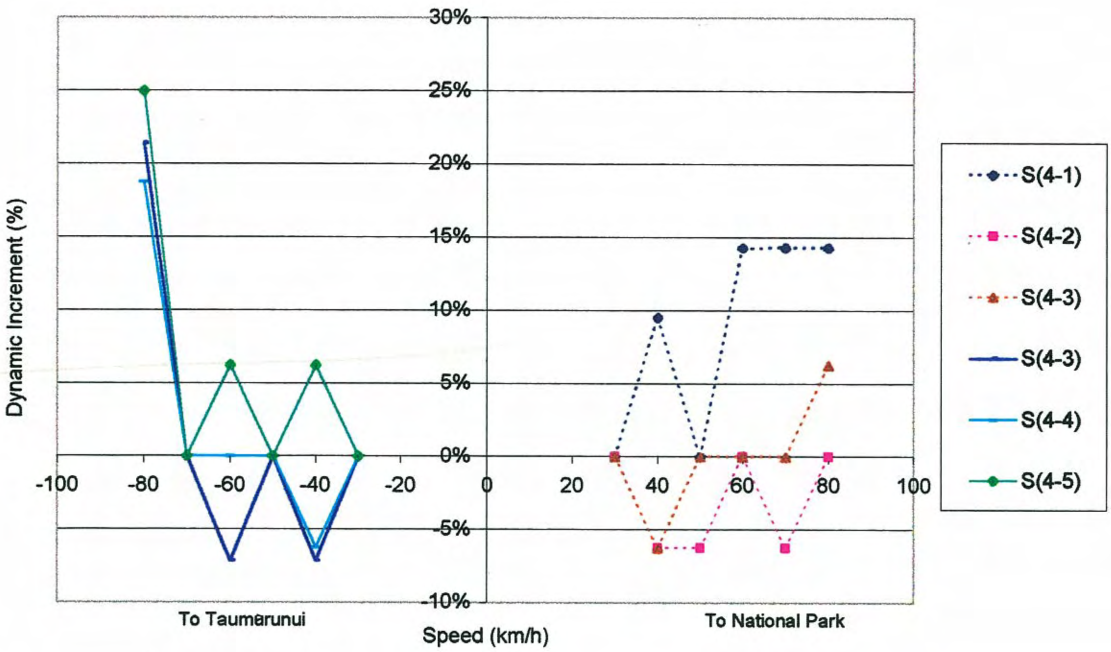


Figure 4.17 Dynamic increment plot for known vehicle.

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(4-1)	95	95	180
S(4-2)	63	71	125
S(4-3)	67	71	130
S(4-4)	75	75	145
S(4-5)	79	87	170
S(4-3A)	51	55	100
SV1(4-3E)	–24	–24	–50
SV2(4-3E)	16	16	35

5. Fitness for Purpose Evaluation

5.1 Main Girders

The structural assessment described in section 3.2 of this report indicated that web shear was the critical mode of failure for the Wanganui Bridge. The Fitness for Purpose Evaluation of the superstructure was for both bending and shear.

5.2 Girder Bending

5.2.1 Multiple Presence

The Wanganui 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 one instrumented 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 and has been used in this report. The method may be conservative because it assumes that a maximum event occurs in each lane at the same time.

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

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 the Bridge Manual. The distributions of strain presented in Figure 5.1 are based on the distribution factors from the known vehicle and the extrapolated health monitoring data. The diagram shows a transverse distribution of strain for each direction and the sum of these two distributions. This has been completed for both the 95% in 1 year (serviceability) event and the 95% in 100 year (ultimate) event. The data show that the highest strain related to a multiple presence event occurs in Girder 3. The maximum strains include 240 $\mu\varepsilon$ for the serviceability limit state (95% in 1 year) and 275 $\mu\varepsilon$ for the ultimate limit state (95% in 100 years). Also on this diagram is the limiting strain for each of these cases. These strains are derived in section 5.1.2 of this report. The actual strains that would be created from the multiple presence events are well below these limiting strains.

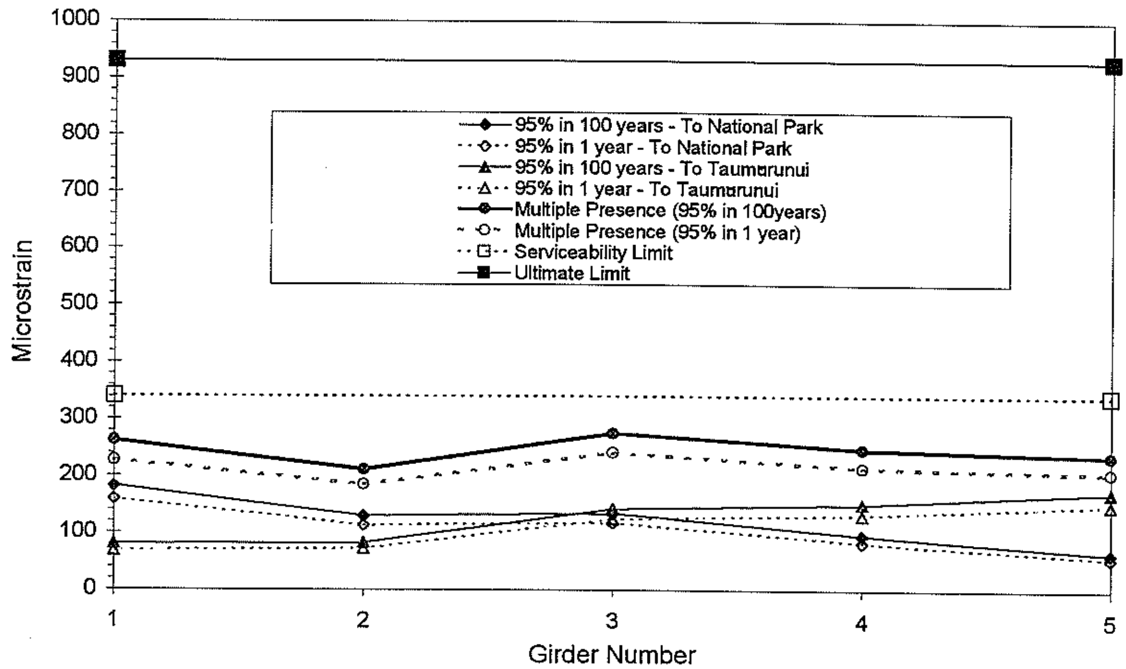


Figure 5.1 Multiple presence effects using the approach that is consistent with the Bridge Manual.

5.2.2 Moment versus Strain Relationship

Figure 5.2 illustrates a theoretical moment versus strain curve for a typical girder of the Wanganui Bridge. The graph presents the method used by Infratech to obtain a relationship between bending moment and strain in the soffit of the girder. Because the girders have not cracked in service, this relationship is presented for the girder behaving as an uncracked linear elastic section, even at ultimate capacity. This relationship is, however, a conservative approximation, and 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 would crack, and at which the behaviour of the girder would normally change. However, as the relationship is presented as a linear elastic relationship, the behaviour remains linear. The line BCD represents the linear elastic representation of the behaviour to ultimate bending strength.

Superimposed on Figure 5.2 is the reduced ultimate moment capacity and the service load capacity, and the corresponding strains. The service load capacity corresponds to the decompression moment, the calculation of which assumes post-tensioning losses of 30%. The dead load, factored dead load, and the corresponding strains are also illustrated, as are the ultimate multiple presence event and the service load multiple presence event represented by shaded rectangles. The diagram shows some capacity reserves for the service load case and significant capacity reserves for the ultimate load case.

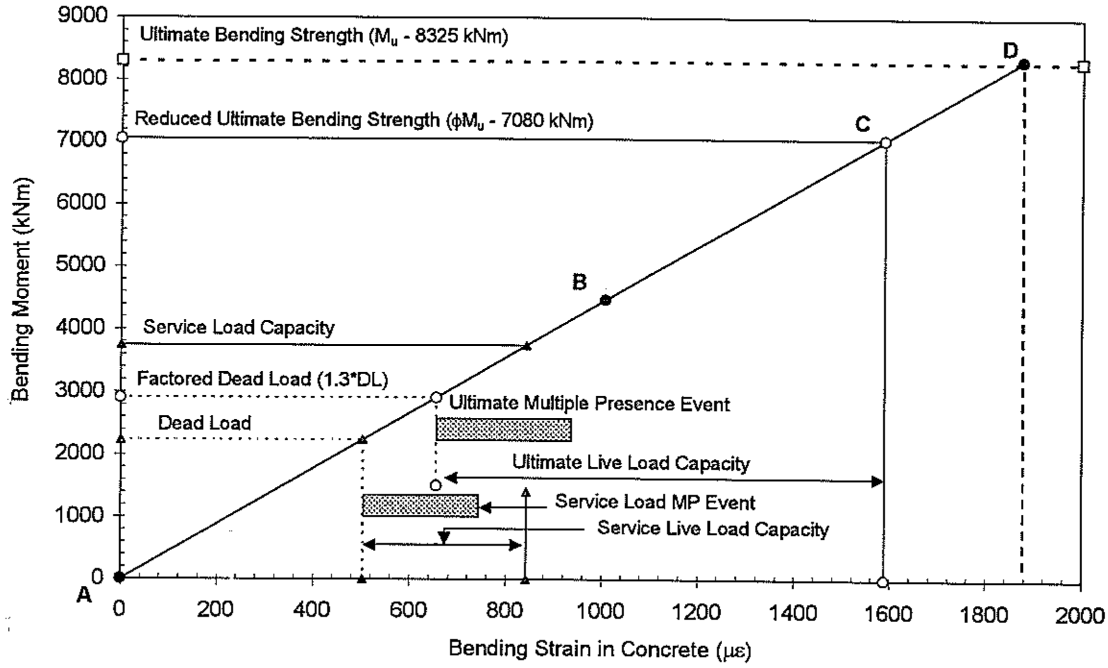


Figure 5.2 Moment versus strain relationship, and summary of Fitness for Purpose for Girder 1.

5.2.3 Fitness for Purpose Evaluation

The Fitness for Purpose Evaluation for midspan bending for the ultimate limit state and the serviceability limit state for Girder 3 are listed in Tables 5.1 and 5.2.

Table 5.1 summarises the calculation of the Fitness for Purpose Evaluation based on the ultimate bending strength of the girder. 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 ultimate strength for this bridge is 340%. This evaluation compares poorly with the theoretical 0.85 HO rating evaluation (155%) and the 0.85 HN posting evaluation (165%) (see Table 3.3). The comparison with the 0.85 HN loading is the most appropriate as this evaluation is related to actual heavy vehicle traffic.

The Fitness for Purpose Evaluation based on serviceability is presented in Table 5.2. This method of calculating the evaluation is based on the moment that will cause decompression in the concrete at the soffit of the girder. The multiple presence event for the 95% in 1 year event, as illustrated in Figure 5.1, is 240 με (for Girder 3). This gives a Fitness for Purpose Evaluation based on the serviceability limit state that is equal to 140%. This compares poorly with the theoretical rating evaluation of 85% and the posting evaluation of 110% for this bridge.

Table 5.1 Summary of Fitness for Purpose Evaluation based on the ultimate bending strength of Girder 3.

Item	Result
Ultimate Strength (ϕM)	7080 kNm
Dead Load (*1.3)	2905 kNm
Ultimate Live Load Capacity Moment – Ultimate ($\gamma_o R_o$)	4175 kNm
Ultimate Live Load Capacity – Equivalent Strain ($\gamma_o R_o$)	930 $\mu\varepsilon$
Ultimate Traffic Load Effect (Multiple Presence)	275 $\mu\varepsilon$
Fitness for Purpose Evaluation	340%

Table 5.2 Summary of Fitness for Purpose Evaluation based on the serviceability limit state of Girder 3.

Item	Result
Serviceability Strength (M_o)	3740 kNm
Dead Load	2235 kNm
Ultimate Live Load Capacity Moment – Serviceability ($\gamma_o R_o$)	1505 kNm
Serviceability Live Load Capacity – Equivalent Strain ($\gamma_o R_o$)	340 $\mu\varepsilon$
Serviceability Traffic Load Effect – 95% in 1 year (Multiple Presence)	240 $\mu\varepsilon$
Fitness for Purpose Evaluation	140%

5.3 Girder Shear

As outlined in section 3.3 of this report, the critical failure mode for this structure is web shear and this mode will determine the final rating of the structure.

The extrapolated results from the two transducers that measured web shear effects were –50 $\mu\varepsilon$ (SV1(4-3E)) and 35 $\mu\varepsilon$ (SV2(4-3E)). The Multiple Presence effects were also considered for the shear in Girder 3, and an analysis similar to that illustrated in Figure 5.1 gave strains of –100 $\mu\varepsilon$ and 75 $\mu\varepsilon$ respectively.

A summary of the Fitness for Purpose Evaluation based on the web shear is presented in Table 5.2, in which the results are presented in terms of a shear stress (τ). The method is based on Mohr's circle (Warner & Faulkes 1989), and assumes that the transducers were positioned at the centroid. Also results presented in the table are for concretes having 21 MPa and 40 MPa compressive strengths.

Because the results from the two strain gauges are not equal and opposite, this indicates that the transducers were not positioned at the centroid of the section. Therefore some error may have been made in calculating web shear from these results. As outlined in section 4.1 of this report, three strain gauges arranged in a rosette pattern are required to accurately measure web shear effects. However these results confirm the theoretical analysis which showed that web shear is an issue with this structure, and that the results are sensitive to the concrete strength. The analysis also assumes that the centre girder is critical in shear. This assumption is based on the fact that results of the Health Monitoring indicated that the centre girder is the critical girder in bending.

Table 5.3 Summary of Fitness for Purpose Evaluation for web shear in Girder 3.

Item	Result (21 MPa Concrete)	Result (40 MPa Concrete)
Ultimate Strength (ϕV_n)	2.7 MPa	3.0 MPa
Dead Load (*1.3) – Equivalent Stress	0.9 MPa	0.9 MPa
Ultimate Live Load Web Shear Capacity ($\gamma_o R_o$)	1.8 MPa	2.1 MPa
Ultimate Traffic Load Effect SV1(4-3E) (Multiple Presence)	–100 $\mu\varepsilon$	–100 $\mu\varepsilon$
Ultimate Traffic Load Effect SV2(4-3E) (Multiple Presence)	75 $\mu\varepsilon$	75 $\mu\varepsilon$
Equivalent Ultimate Traffic Load Effect – Stress (Multiple Presence)	2.0 MPa	2.0 MPa
Fitness for Purpose Evaluation	85%	100%

5.4 Summary

The theoretical analysis of this bridge indicated that the web shear capacity was limiting the capacity of this bridge. The rating evaluation based on web shear ranged from 45% to 65% based on concrete strengths of 21 MPa and 40 MPa respectively. The analysis also indicated that serviceability criteria (limiting stress across the construction joints at midspan) was limiting the rating evaluation to 85%. In both of these analyses the edge girder was the critical component of the structure because of the eccentricity of loading, including the positioning of the 0.85 HO loading close to the kerb.

The Health Monitoring programme indicated that, while the edge girders were critical for single vehicle events, the centre girder became critical when multiple presence effects were considered.

The results from the Health Monitoring programme and the Fitness for Purpose Evaluation showed:

- The Fitness for Purpose Evaluation, considering the limiting of tensile stress across the construction joints at midspan, was 140%.

- The Fitness for Purpose Evaluation for web shear ranged from 85% to 100% for concrete strengths of 21 MPa and 40 MPa respectively.

These results are higher than the theoretical evaluations because:

- The actual response of the bridge was less than the predicted response.
- The distribution of load from the actual traffic was more evenly distributed through the five girders than calculated for the theoretical analysis. This is mainly related to the differences between the loading required by the Bridge Manual for evaluation, and the loading induced by the actual heavy traffic on this bridge.

6. Conclusions

This report presents the details and results of the Health Monitoring programme applied to one of the 30.5 m spans on the Wanganui Bridge. A Fitness for Purpose Evaluation has also been derived for the bridge, based on Health Monitoring.

Theoretical Analysis
The results of the evaluations based on flexural capacity are greater than those for web shear. Consequently, the web shear capacity of the main girders is the critical issue with this bridge, and determines the capacity of the superstructure. The theoretical rating rating evaluation based on the web shear is 45% and the posting evaluation of 55%, assuming that 21 MPa strength concrete was used. The rating evaluation is 65%, assuming use of 40 MPa concrete. The rating evaluation from the 1999 TNZ Structural Inventory is 61%. It is understood that, according to normal practice, that this bridge may require posting.

The serviceability limit state (with no tensile stress across construction joints) also limits the bending capacity of the structure. The rating and posting evaluations based on this criterion are 85% and 110% respectively. The performance of the deck was not considered in this report.

Health Monitoring Results
The findings for the Health Monitoring of the Wanganui Bridge are that:

- The ambient heavy vehicle traffic is inducing bending moments in the bridge that are similar to those induced by the 0.85 HN vehicle; and very little overloading occurs on this route.
- The recorded strains are significantly lower than the strains predicted in the analysis. This may be related to bearing restraint or to differences in actual and assumed material properties.
- The dynamic effects on this bridge are similar to those predicted by the impact factor from the Bridge Manual.
- The load distribution across the five girders is more uniform for the actual heavy traffic effects compared to the distribution assumed for rating and posting evaluation loads. This contributes to higher evaluations if they are based on the health monitoring data.

Fitness for Purpose Evaluation
The Fitness for Purpose Evaluation based on web shear ranged from 85% to 100% for concrete strengths of 21 MPa and 40 MPa respectively.

If the serviceability limit state, which limits the tensile stresses across the interfaces between segments, is considered, the Fitness for Purpose Evaluation would be 140%.

Both the theoretical and Health Monitoring results indicate that web shear is an issue with this bridge.

7. Recommendations

The recommendations therefore are that:

- Web shear effects should be measured more accurately using three strain gauges (rosettes) at a number of locations.
- Shear results between the individual girders should be compared to determine the critical girder that is in shear.
- The actual concrete strength should be measured, because the web shear strength is sensitive to the concrete strength, and in particular to the principal tensile strength.
- Further research into the web shear capacity of prestressed concrete bridge girders is warranted.
- The results from the Health Monitoring programme showed that the girders of the Wanganui Bridge may have enough shear capacity to avoid the requirement to post it, but further investigations are needed.

8. References

AUSTROADS 1992. AUSTROADS Bridge Design Code. AUSTROADS Inc., Sydney.

Standards New Zealand (SNZ) 1995. Concrete structures. *NZS 3101: Part 1 1995.* Standards New Zealand, Wellington, New Zealand.

Transit New Zealand (TNZ) 1994. *Bridge Manual.* Transit New Zealand, Wellington, New Zealand.

Transit New Zealand (TNZ) 1995. *Overweight Permit Manual.* 1st edition. Transit New Zealand, Wellington, New Zealand.

Transit New Zealand (TNZ) 1999. *Structural Inventory.* Database, Transit New Zealand, Wellington, New Zealand.

Turkstra, C.J., Madsen, H.O. 1980. Load combinations in codified structural design. *Journal of the Structural Division, ASCE, 106 (ST12):* 2527 – 2543.

Warner, R.F., Rangan, B.V., Hall, A.S. 1989. *Reinforced Concrete.* Australian Concrete Institute (ACI), 3rd Edition, Longman Cheshire, Melbourne.