# Health Monitoring of Superstructures of New Zealand Road Bridges:

## Waipara Bridge, Canterbury

Transfund New Zealand Research Report No.168

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## Waipara Bridge, Canterbury

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

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

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## Executive Summary


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 Waipara Bridge, on State Highway 1S, crosses the Waipara River about 60 km north of Christchurch, Canterbury Region, South Island. It was selected as one of these ten because it is a prestressed concrete structure, with a conventional strength evaluation that indicates adequate strength. It has been in service since 1971. It is also typical of a large population of prestressed concrete bridges in New Zealand, and has significant value in terms of both replacement costs and transport economics.

The report details a theoretical assessment of the bridge to determine both the critical elements for the Health Monitoring programme, and the Fitness for Purpose Evaluation for the bridge based on the health monitoring data.

*Theoretical Analysis*
The theoretical assessment of the superstructure found that the 0.85 HO* rating evaluation is 110%, the 0.85 HN* posting evaluation is 130%, and that the Deck Capacity Factor (DCF) is 1.39.

The theoretical analysis of the bridge found that the critical issue associated with this bridge's performance is the web shear capacity of the main girders. The rating evaluation based on this failure mode is 110%. This failure mode was not evaluated as part of the Health Monitoring programme.

The capacity of the bridge was assessed for both the ultimate and serviceability limit states for midspan bending. However as this bridge has sufficient capacity to withstand loads for which the bridge has been evaluated, the Health Monitoring programme focused on evaluating the overall performance of the bridge and the effect on the bridge of the heavy vehicle traffic using this route.

*Health Monitoring Results*
The findings from the Health Monitoring programme include:

- The ambient heavy vehicle traffic is inducing bending moments in the bridge that are 50% higher than the known heavy vehicle. Given that the known heavy vehicle produced similar effects to the evaluation (0.85 HN) vehicle, significant overloading occurs on this route, particularly with the northbound traffic.


* HO Highway overweight vehicles; HN Highway normal vehicles

- The recorded strains and deflections at midspan are significantly lower than the theoretically predicted strains. This is probably related to variations in vehicle position on the bridge, and bearing restraint effects.
- The uneven road profile on the southern approach to the bridge may be increasing the effect of vehicles on the bridge.

*Fitness for Purpose Evaluation*

- The Fitness for Purpose Evaluation, based on midspan bending, for the serviceability limit state of this bridge is 320%, and for the ultimate limit state is 440%. These evaluations are significantly higher than the theoretical evaluations. The higher Fitness for Purpose Evaluations are expected to relate mainly to bearing restraint effects, to the influence of the lateral position of vehicles on the bridge, and to bridge and vehicle dynamic effects.
- The theoretical evaluation for the posting load, based on midspan bending, for the serviceability limit state is 235%, and for the ultimate limit state is 155%.
- The posting evaluations are the most relevant to apply to Health Monitoring as they are based on loads which are more representative of actual traffic.
- The Fitness for Purpose Evaluation based on midspan bending cannot be used directly as a means of bridge assessment.
- Girder web shear governs the capacity of the Waipara Bridge, and this was not monitored during the Health Monitoring programme. In addition there is only limited understanding of girder web shear capacity, when factors such as bearing restraint become significant.

## Recommendations

- Web shear effects should be measured, as well as midspan bending effects, when health monitoring this type of prestressed concrete bridge;
- Issues associated with the bearing behaviour on this particular bridge should also be investigated.
- More research is required concerning shear capacity of concrete members, when factors such as bearing restraints are significantly influencing member behaviour.

# 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 Waipara Bridge, on State Highway 1S, crosses the Waipara River about 60 km north of Christchurch, Canterbury Region, South Island. It was selected as one of these ten because it is a prestressed concrete structure, with a conventional strength evaluation that indicates adequate strength. It has been in service since 1971, and is typical of a large population of prestressed concrete bridges in New Zealand. It has a significant replacement value as part of New Zealand's transport infrastructure. The Fitness for Purpose Evaluation indicates that web shear capacity of the main girders governs the capacity of the bridge.

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

*North Island*
Tuakopai Bridge SH29
Tuakopai Stream, Bay of Plenty Region
*TNZ RR No. 174*

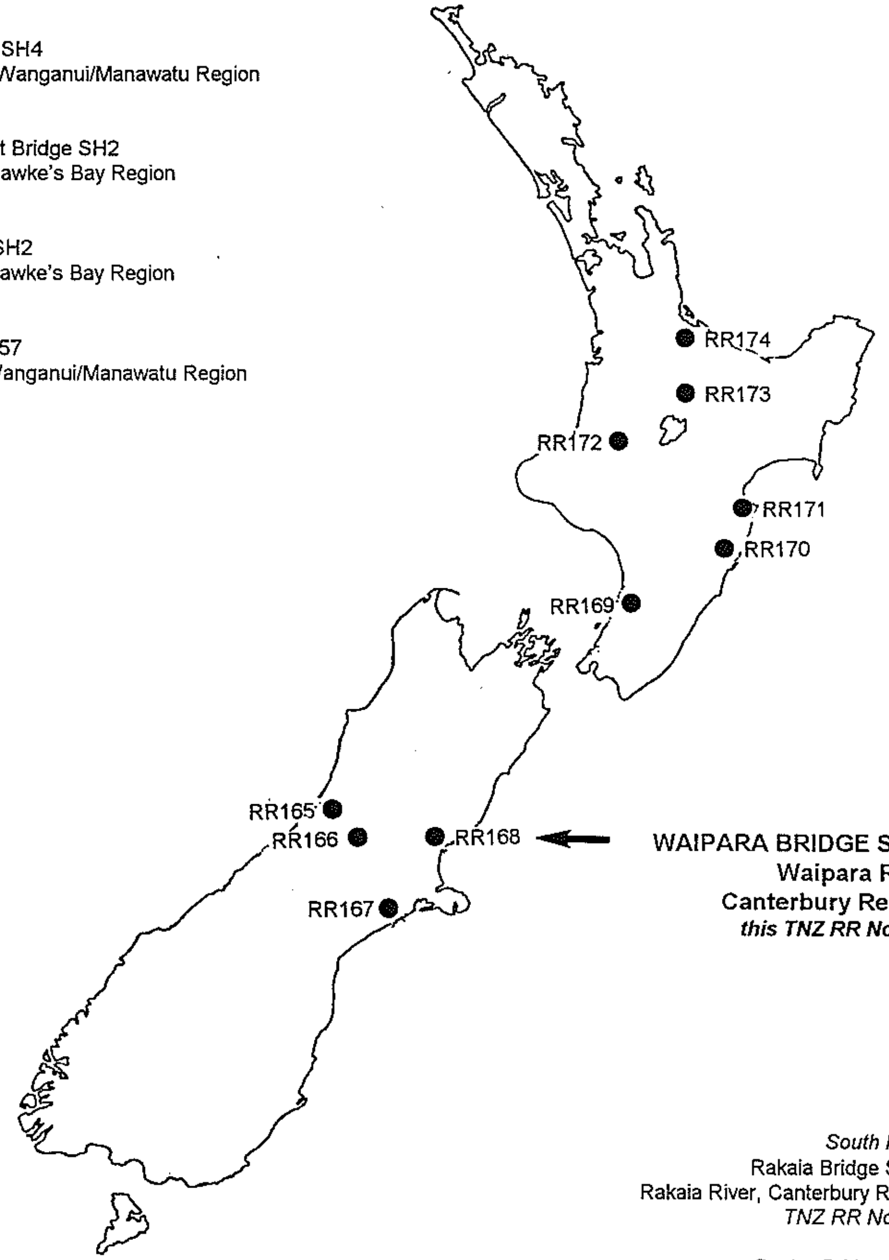
Atiamuri Bridge SH1N
Waikato River, Waikato Region
*TNZ RR No. 173*

Wanganui Bridge SH4
Wanganui River, Wanganui/Manawatu Region
*TNZ RR No. 172*

Waitangi Washout Bridge SH2
Tutaekuri River, Hawke's Bay Region
*TNZ RR No. 171*

Waipawa Bridge SH2
Waipawa River, Hawke's Bay Region
*TNZ RR No. 170*

Otauru Bridge SH57
Otauru Stream, Wanganui/Manawatu Region
*TNZ RR No. 169*




**WAIPARA BRIDGE SH1S**
**Waipara River**
**Canterbury Region**
***this TNZ RR No. 168***

*South Island*
Rakaia Bridge SH1S
Rakaia River, Canterbury Region
*TNZ RR No. 167*

Bealey Bridge SH73
Waimakariri River, Canterbury Region
*TNZ RR No. 166*

Big Wainihinihi Bridge SH73
Big Wainihinihi River, West Coast Region
*TNZ RR No. 165*

*(TNZ RR Transfund New Zealand Research Report)*

# 1. Introduction

## 1.1 Bridge Health Monitoring

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

This project was conducted in 1998-1999 to develop procedures and guidelines for determining the Fitness for Purpose of New Zealand bridges using Health Monitoring technology. Bridge Health Monitoring requires a merging of specifically designed instrumentation technology and data processing, with conventional bridge theory and evaluation techniques. It has not been previously used in New Zealand as a systematic bridge evaluation technique, and consequently a project was conceived with the following objectives:

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

The project 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 Waipara Bridge, on State Highway (SH)1S, which crosses the Waipara River about 60 km north of Christchurch, Canterbury Region, South Island of New Zealand (Figure 1.1). The reasons for choosing this bridge for the representative sample were:

- It is a two-laned, prestressed concrete structure;
- It has a conventional strength evaluation that indicates adequate strength;
- It has been in service since 1971;
- It is also typical of a large population of prestressed concrete bridges in New Zealand; and
- It is a relatively large structure, and so has a high replacement value. It is also on a major transport route, and has significant value in terms of transport economics.

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

## 1.2 Applying Health Monitoring Technology

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

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

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

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

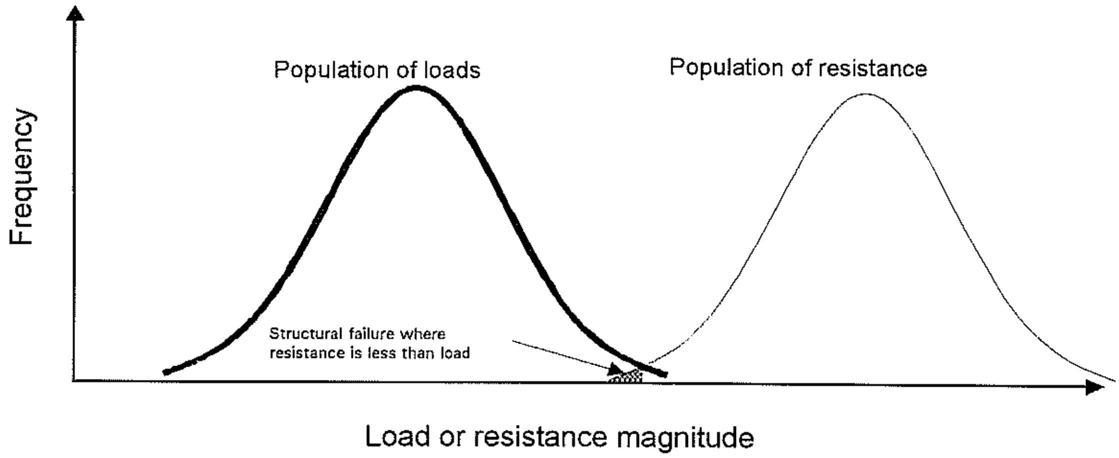



**Figure 2.1 Statistical representation of structural failure.**

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, although 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 and decks of a bridge separately. The evaluation approach described in Section 6 of the Manual is summarised here.

### 2.3.1 Main Members

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

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

where:

| | | | |
|---|---|---|---|
| $R_o$ = | Overload Capacity | $DL$ = | Dead Load Effect |
| $\phi$ = | Strength Reduction Factor | $\gamma$ = | Load factors on other effects |
| $R_I$ = | Section Strength | $\gamma_o$ = | Overload Factor |
| $\gamma_D$ = | 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.

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

#### 2.3.1.2 *Posting Evaluations*

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

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

### 2.3.2 Decks

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

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

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

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

## 2.4 The Health Monitoring Approach

### 2.4.1 Theory of this Approach

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

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

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

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

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

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

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

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

$$\gamma^{o} R^{o} = \phi R^{i} - \gamma^{D}(DL) - \sum(\gamma(Other\ \ Effects)) \qquad \text{(Equation 5)}$$

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

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

where:

$FPE$ = Fitness for Purpose Evaluation
$\gamma_o R_o$ = Ultimate Traffic Live Load Capacity
$UTL\ Effect$ = Ultimate Traffic Load Effect derived from health monitoring data

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

### 2.4.2 Behavioural Test using a Known Vehicle

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

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

Behavioural tests using a known vehicle were conducted at the Waipara Bridge during the Health Monitoring programme, and the results are given in section 4.4 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 2.2 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 (nomenclature as in the Bridge Manual), taken from the drawings and used in the analysis, are as follows:

- Concrete Girders $f'_c$ = 38 MPa, E = 27 400 MPa
- Prestressing Cables $f_y$ = 1860 MPa, E = 195 000 MPa

### 3.2.1 Girder Bending

The maximum bending moment in the girders resulting from the dead load is 890 kNm/girder. The dead load does not include the additional load of the guardrails on the edge girders. A summary of the maximum bending moments in a typical edge girder resulting from the various loads applied to the grillage model is presented in Table 3.1. The results in the table are not factored, and they represent the maximum bending moment in the girder with the vehicle at the greatest allowable eccentricity. The edge girder is the critical girder in the structure.

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

| Load | Bending Moment (kNm) |
|---|---|
| Dead Load | 890 |
| Known Vehicle | 445 |
| 2x 0.85HN Vehicles (Posting Load) | 805 |
| 0.85HO + 0.85HN Vehicles (Rating Load) | 1155 |

The bending capacity ($\phi$M) of the concrete girders in the superstructure, calculated in accordance with Section 16 of the Concrete Structures Standard (NZS 3101: Part 1 1995), is 4135 kNm. This strength does not include any contribution from the kerbs or the guardrails.

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

The capacity of a typical girder based on the serviceability condition is 3305 kNm. This capacity is based on a maximum tensile stress in the concrete in the soffit of the girder equal to $0.5\sqrt{f'_c}$, as described in Table 16.2 of the Concrete Structures Standard (NZS 3101: Part 1 1995).

### 3.2.2 Girder Shear

The maximum shear force in a typical edge girder obtained from the grillage analysis is presented in Table 3.2. The edge girders are also the critical girders in shear.

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

| Load | Shear Force (kN) |
|---|---|
| Dead Load | 190 |
| Known Vehicle | 105 |
| 2x 0.85HN Vehicles (Posting Load) | 150 |
| 0.85HO + 0.85HN Vehicles (Rating Load) | 230 |

The shear capacity of the main girders was calculated in accordance with Section 9 of the Concrete Structures Standard (NZS 3101: Part 1 1995). The flexural shear capacity ($\phi V$) is 775 kN.

The web shear was also calculated using Section 9 of the Concrete Structures Standard. The shear capacity of the web ($\phi V$) is equal to 725 kN, and the critical failure mode in terms of shear is web shear.

### 3.2.3 Deck Capacity

The capacity of the deck was not considered in this report. The Deck Capacity Factor (DCF) for the deck based on the TNZ Structural Inventory is 1.39. This indicates that the deck capacity is satisfactory for the required traffic loadings.

## 3.3 Theoretical Load Evaluation

The process required to determine the theoretical load evaluations 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. The rating has been assessed for the bending and shear in the girders. The table also presents a comparison of the load evaluation calculated by Infratech Systems & Services (Infratech), and the evaluation recorded in the 1999 TNZ Structural Inventory.

Values of 1.27 for the impact factor and of 1.3 for the dead load factor were used in calculating the evaluations. The rating loads presented in Table 3.3 do not include impact factors, but these are included in the rating and posting evaluation calculations.

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

| Mode of Failure | $\phi$ Ultimate Capacity | 0.85 HO Rating Load | 0.85 HN Posting Load | Dead Load | 0.85 HO Rating (Infratech) | 0.85 HN Posting (Infratech) | Rating (Structural Inventory) |
|---|---|---|---|---|---|---|---|
| Girder Bending (Ultimate) | 4135kNm | 1155kNm | 805kNm | 890kNm | 135% | 155% | |
| Girder Bending (Service) | 3305kNm | 1155kNm | 805kNm | 890kNm | 165% | 235% | 108% |
| Web Shear | 725kN | 230kN | 150kN | 190kN | 110% | 130% | |

The overall rating of the girders is taken as the minimum rating of all the components. For this bridge, the overall evaluation is 110% with web shear being the critical failure mode. This rating evaluation compares well to the rating of 108% which is documented in the TNZ Structural Inventory.

## 3.4 Summary

The grillage analysis of the Waipara Bridge was used to determine the bending moment and shear in the girders of a typical span, based on various vehicle loadings. It showed that the performance of the superstructure is satisfactory, based on the Bridge Manual procedure. Subsequently the Health Monitoring programme focused on evaluating the overall performance of the bridge, and the effect on the bridge of the heavy vehicle traffic using this route.

Web shear is the critical failure mode with the corresponding rating evaluation being 110%.

The Deck Capacity Factor is 1.39 using the TNZ Structural Inventory, but the performance of the deck is not considered in this report.

# 4. Health Monitoring Programme

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

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

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

## 4.1 Instrumentation

The instrumentation installed on the bridge included five Demountable Strain Gauge transducers and three Deflection transducers. The instrumentation was installed on the southernmost span of the structure, and the positions of this instrumentation are illustrated in Figure 4.1.

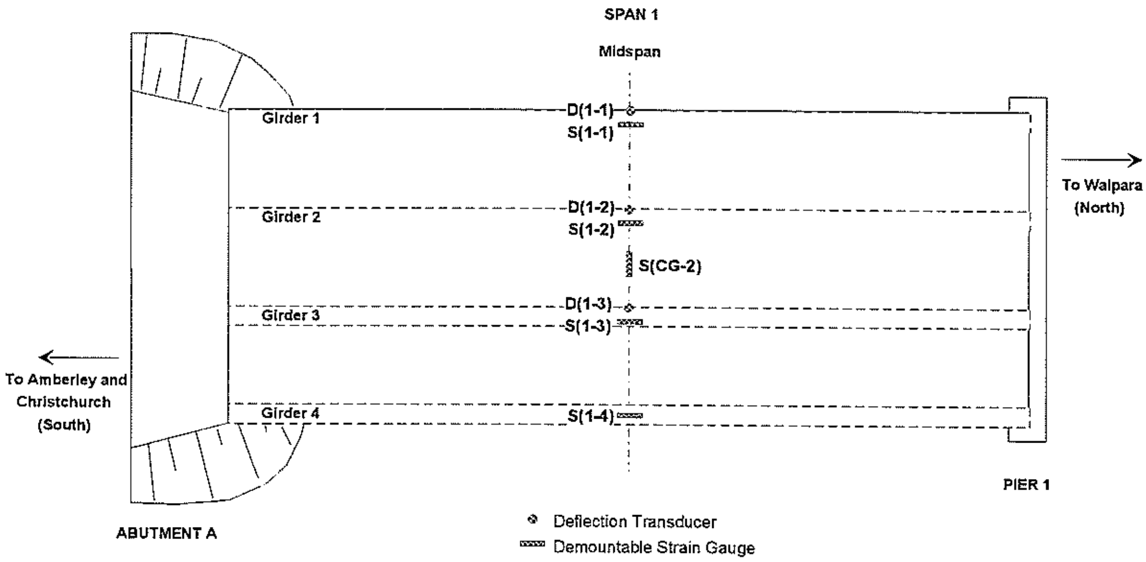



**Figure 4.1 Instrumentation plan for Waipara Bridge.**

Figure 4.2 shows the installation of demountable strain gauge and deflection transducers at midspan, and Figure 4.3 illustrates the cross section of the monitored span detailing the positions of the instrumentation.

**Figure 4.2 Instrumentation on the midspan of Waipara Bridge.**

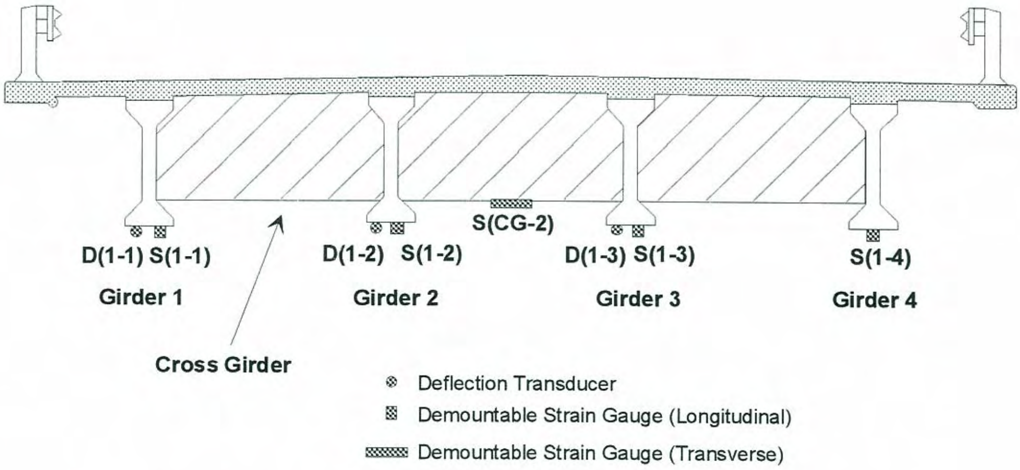



**Figure 4.3 Cross section of bridge with instrumentation (looking north).**

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

## 4.2 Procedure

The health monitoring of the structure began on Tuesday 24 November, and continued until Wednesday 25 November, 1998, giving a total monitoring period of approximately 25 hours. During the one-day monitoring period, the response of the bridge to 481 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 Tuesday 24 November, 1998. The known vehicle was a seven-axled heavy vehicle of known mass and dimensions, supplied by McCarthy Transport Contractors, Raetihi, and shown in Figure 4.4. The axle weights and configuration are illustrated in Figure 4.5, and the gross mass of the vehicle was 44.2 tonnes.

**Figure 4.4 The known vehicle used for behavioural testing.**

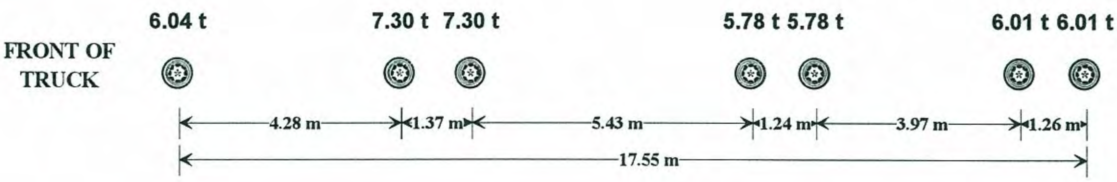



**Figure 4.5 Axle mass and configuration of the known vehicle.**

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 100 km/h, in increments of 20 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 heavy vehicles between the test runs with the known vehicle.

## 4.3 Short-Term Health Monitoring Results

### 4.3.1 Girder Response

A typical strain response versus time was graphed (as waveforms) for the midspan bending strains recorded during the health monitoring for the passage of a typical heavy vehicle. It is presented in Figure 4.6. These waveforms are from one of the larger events recorded. The waveforms show a large degree of dynamic response during and after the vehicle has passed over the instrumented span.

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. 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 presented in Figure 4.7 displays consistently higher responses from transducer S(1-1) vehicles travelling north, compared with those from transducer S(1-4) vehicles travelling south.

The response of the deflection transducers is presented in Figure 4.8. The dynamics of the span are again evident from the deflection response of the individual girders. Girder 4 was not instrumented with a displacement transducer.

The scatter diagram for the deflection transducers is presented in Figure 4.9, and . shows that the highest deflection responses were from Girder 1.

The waveform response for the strain in the cross girder is presented in Figure 4.10. The waveform shows a significant dynamic response in the cross girder due to the ambient heavy traffic. This waveform is one of the larger responses recorded in the cross girder and is of a different event to the other waveforms presented in Figures 4.6 and 4.8. The scatter diagram for the cross girder strains is presented in Figure 4.11.

**Figure 4.6 Strain response versus time for midspan strain transducers of span 1 for event recorded at 6.57am, 25 November 1998 (vehicle travelling north towards Waipara).**

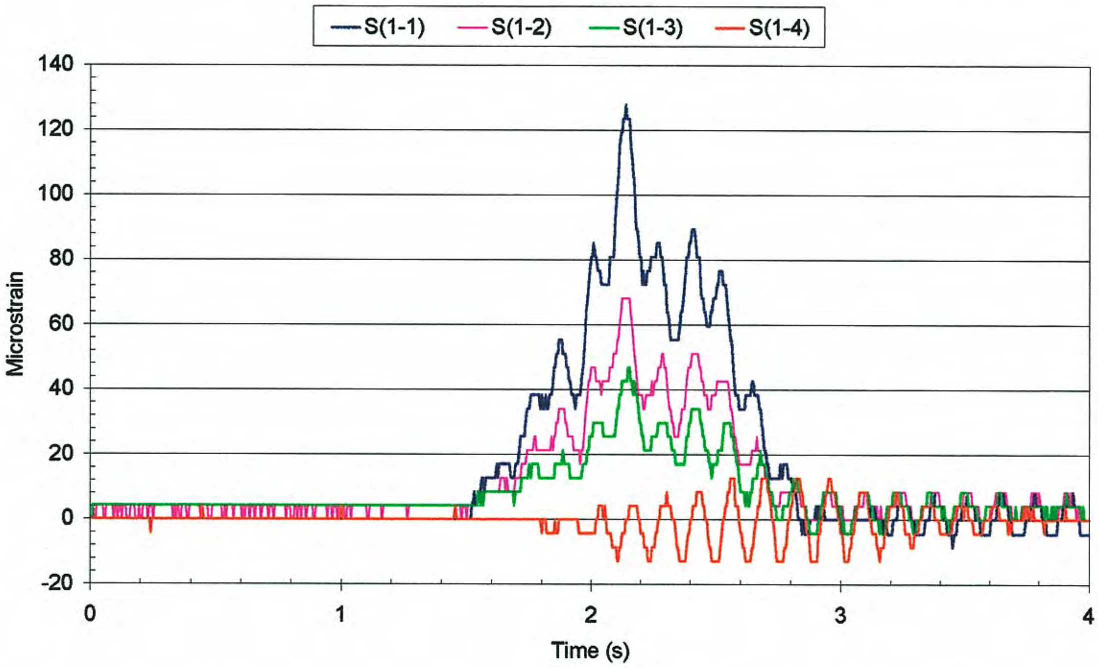



**Figure 4.7 Scatter diagram for midspan bending strains of span 1 for entire monitoring period.**

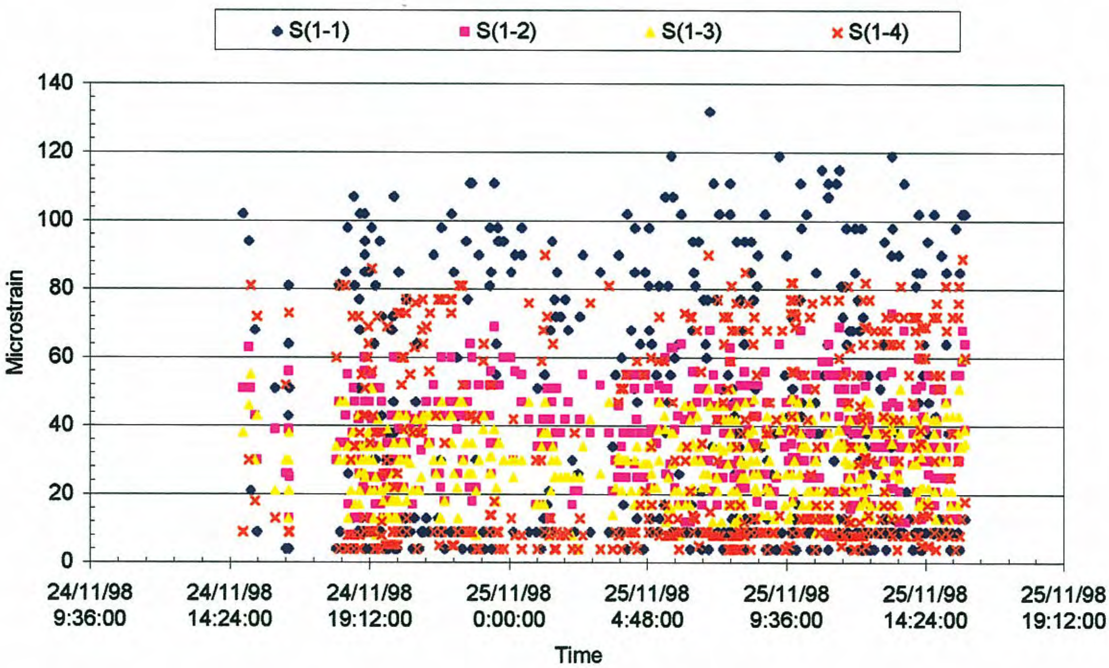


**Figure 4.8 Waveform for transducers measuring the deflection response for event recorded at 6.57am, 25 November 1998.**

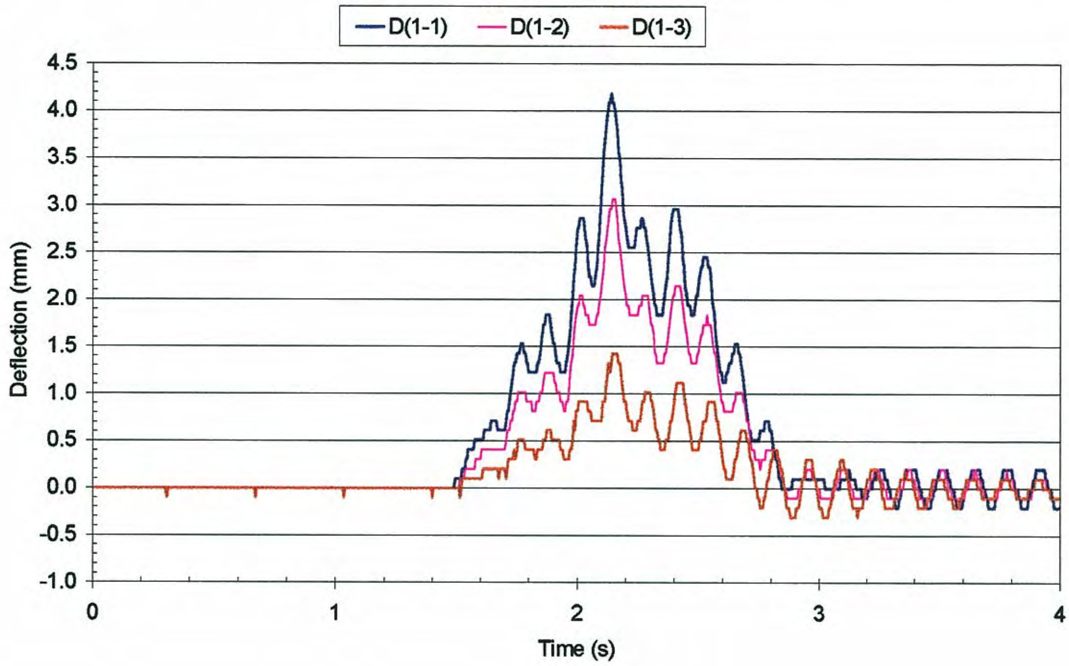



**Figure 4.9 Scatter diagram for midspan deflections.**

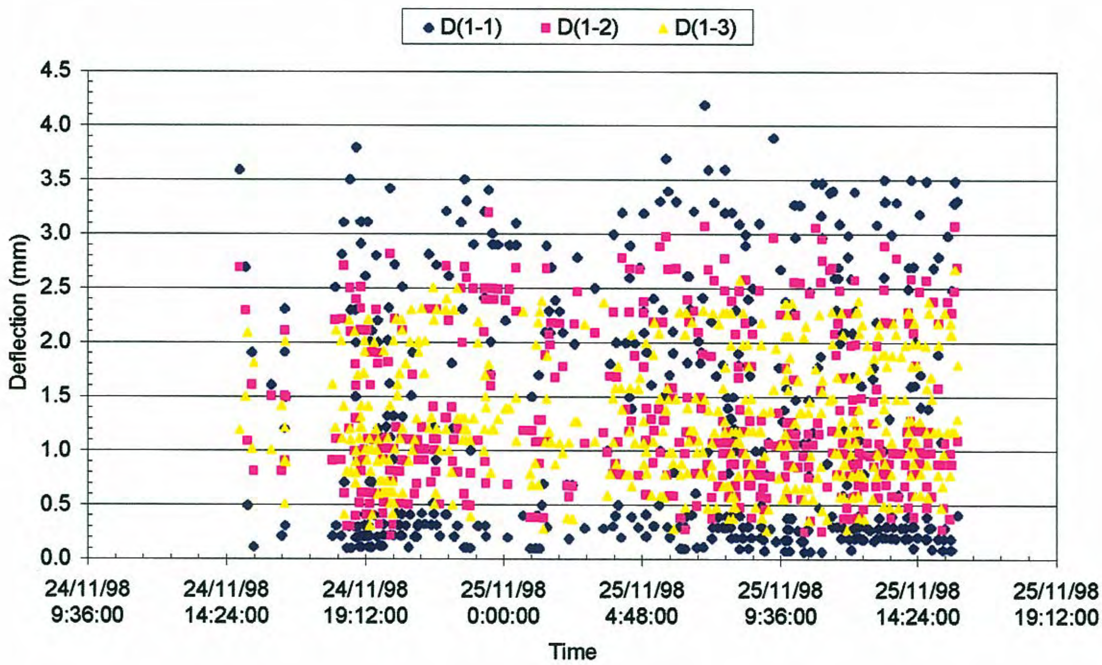


**Figure 4.10 Waveform for the strain in the cross girder between Girders 2 and 3 for event recorded at 9.20am, 25 November 1998.**

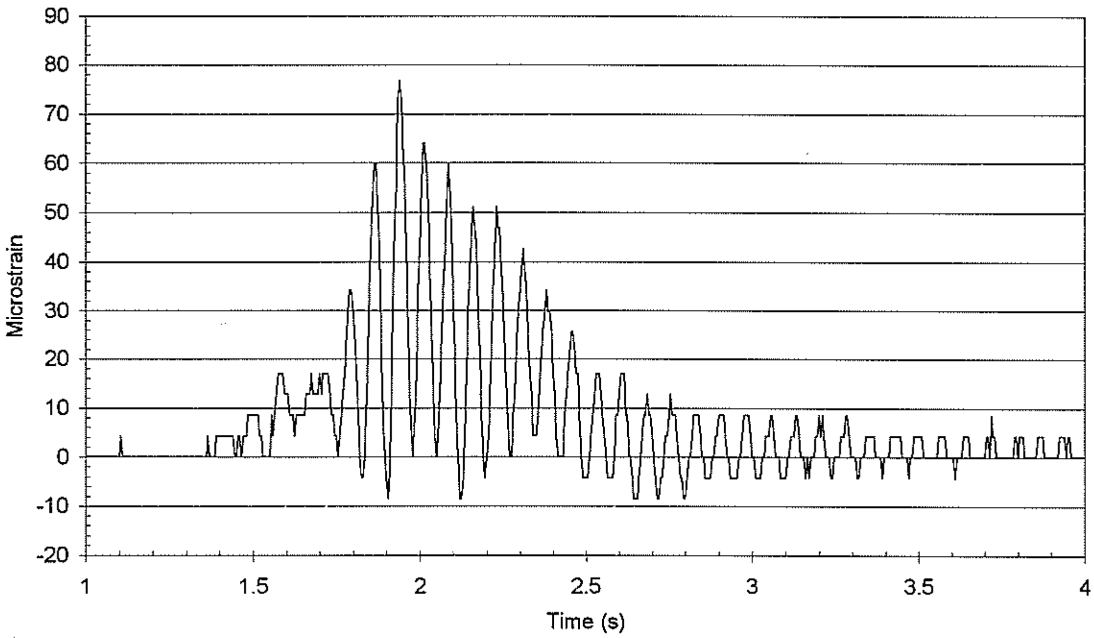
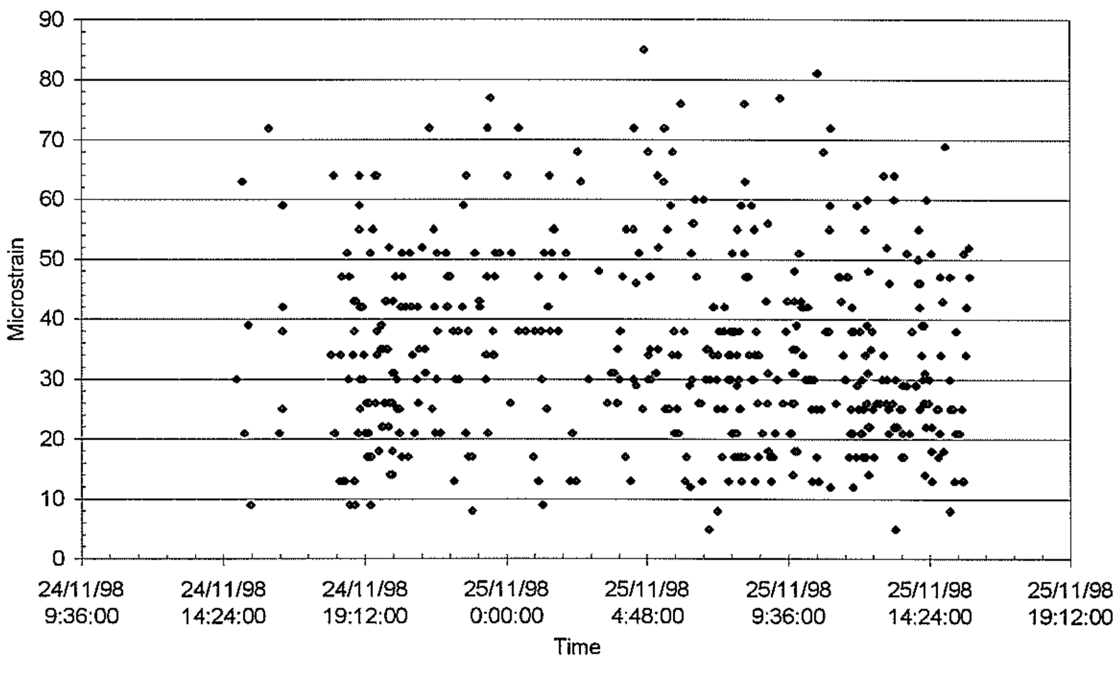



**Figure 4.11 Scatter diagram for transducer measuring strain in the cross girder (span 1) for entire monitoring period.**



### 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(1-4) in Figure 4.12. The histogram illustrates a very large number of samples corresponding to approximately 10 με, and the remaining distribution contains a much smaller number of points. 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 plots for each of the transducers measuring midspan bending strain are presented in Figure 4.13. In this figure the data for the transducers measuring midspan strain in the girders are separated into opposite (north and south) directions. 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 4.12 Histogram and cumulative distribution function for midspan transducer S(1-4).**

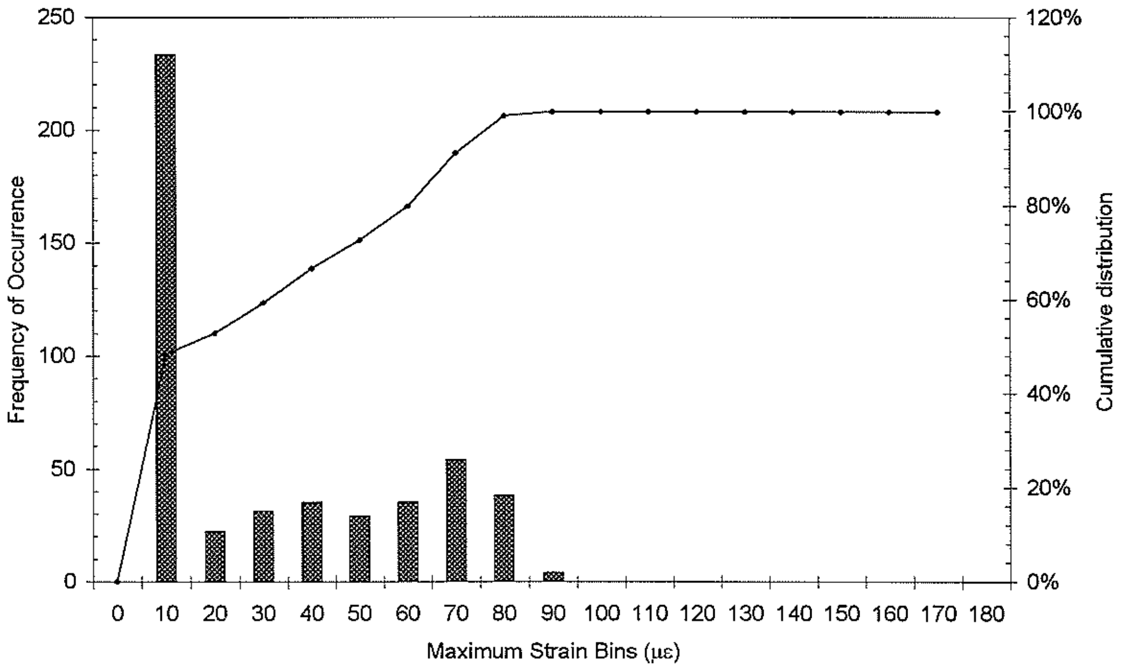


**Figure 4.13 Inverse normal plot for strain transducers installed on span 1 girders at the midspan.**

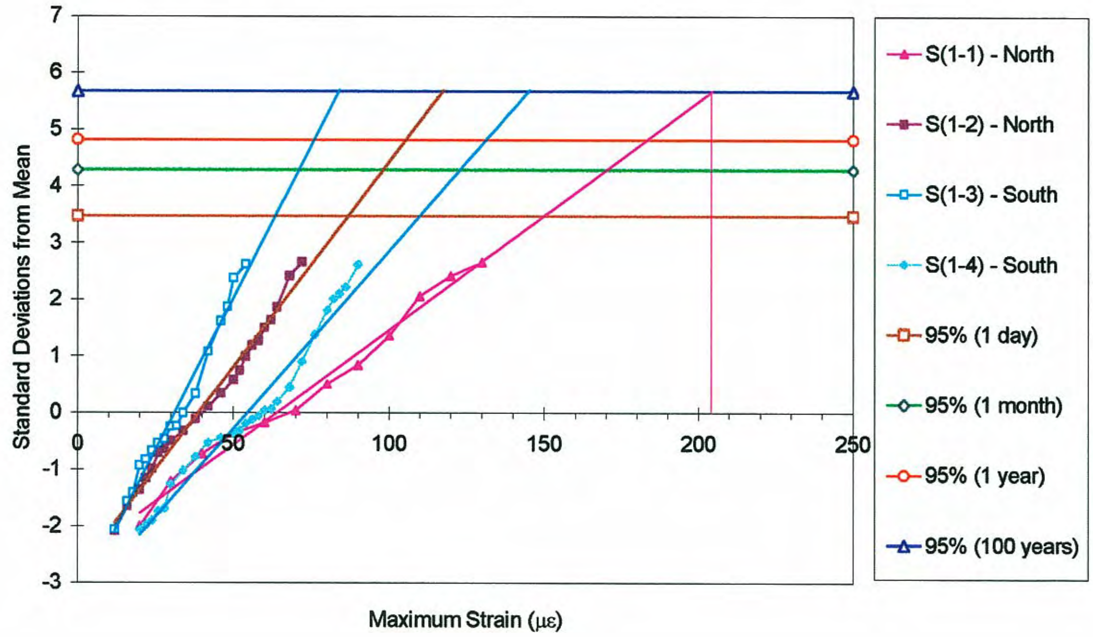
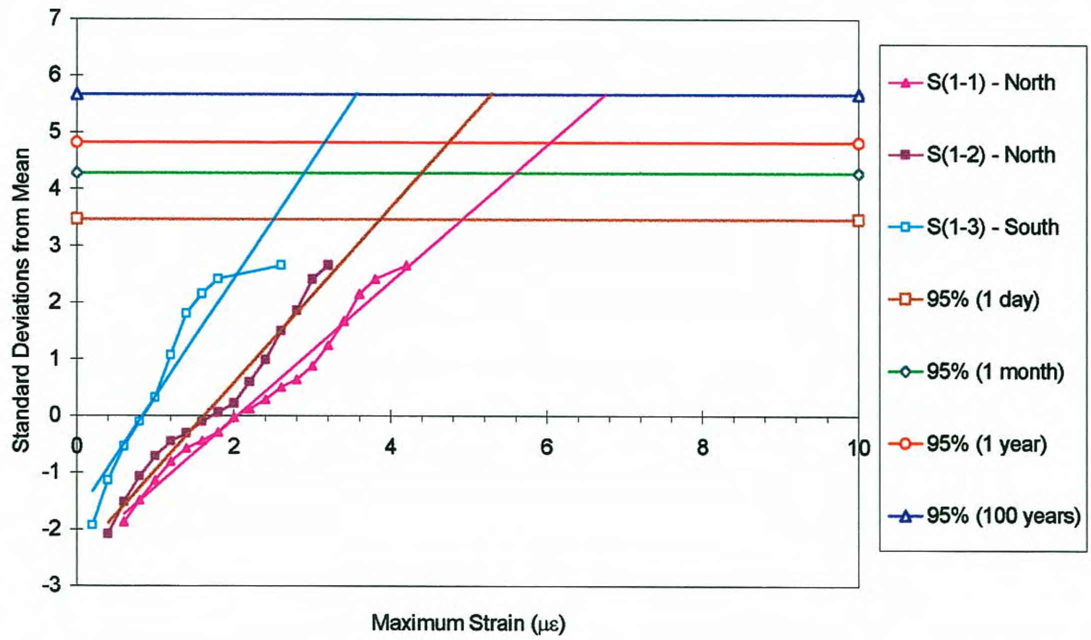



**Figure 4.14 Inverse normal plot for transducers measuring deflection.**

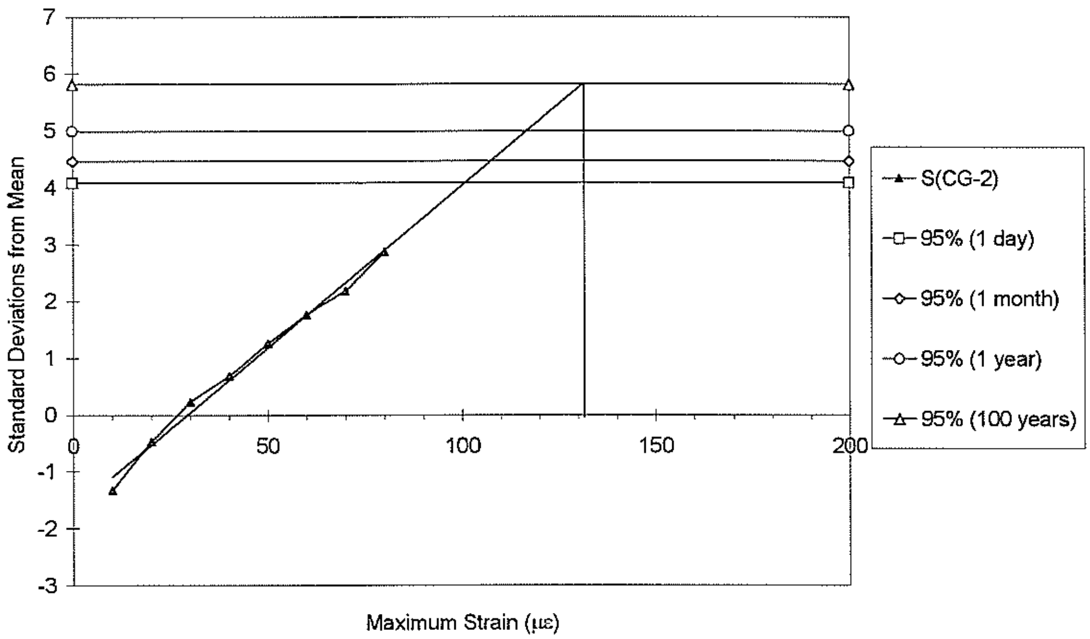


Horizontal lines representing the expected position of the 95% confidence limit for the data for 1 day, 1 month, 1 year, and 100 years have been plotted. Extrapolating the recorded data allows estimates of strain for these longer return intervals. The strain extrapolated for the 95% confidence limit for 100 years represents the ultimate traffic load effect for the Fitness for Purpose Evaluation as outlined in section 2 of this report.

The inverse normal plot shows that the strain extrapolated for the 95% confidence limit for 100 years (ultimate traffic load effect) is the greatest for the midspan transducer S(1-1). The extrapolated value is approximately 205 με. The strains recorded in the edge girders were higher than those recorded in the middle girders.

The inverse normal plots for the deflection transducers installed at the midspan are presented in Figure 4.14, and the extrapolated plot for the strain transducer installed on the cross girder is presented in Figure 4.15.

**Figure 4.15 Inverse normal plot for transducer measuring strain in the cross girder.**



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

**Table 4.1 Extrapolated data obtained from inverse normal distributions.**

| Transducer | Maximum Recorded Value (Health Monitoring) | Extrapolated Value (95% confidence limit) for 100 years |
|---|---|---|
| | *Strains ($\mu\varepsilon$)* | |
| S(1-1) | 130 | 205 |
| S(1-2) | 72 | 115 |
| S(1-3) | 55 | 80 |
| S(1-4) | 90 | 145 |
| S(CG-2) | 85 | 135 |
| | *Deflections (mm)* | |
| D(1-1) | 4.2 | 6.7 |
| D(1-2) | 3.2 | 5.3 |
| D(1-3) | 2.7 | 3.4 |

## 4.4 Known Vehicle Testing

A typical waveform from the testing with the known vehicle (travelling north) is presented in Figure 4.16. The known vehicle testing was performed at different vehicle speeds ranging from a crawl to 100 km/h. The maximum strains for each transducer recorded from the known vehicle are presented in Table 4.2.

**Table 4.2 Maximum responses (strains ($\mu\varepsilon$), deflections (mm)) recorded for known vehicle testing.**

| Transducer | Maximum Response |
|---|---|
| S(1-1) | 85 $\mu\varepsilon$ |
| S(1-2) | 47 $\mu\varepsilon$ |
| S(1-3) | 38 $\mu\varepsilon$ |
| S(1-4) | 77 $\mu\varepsilon$ |
| S(CG-2) | 47 $\mu\varepsilon$ |
| D(1-1) | 2.3 mm |
| D(1-2) | 2.0 mm |
| D(1-3) | 2.0 mm |

**Figure 4.16 Typical waveform for the known vehicle travelling at 100 km/h north towards Waipara.**

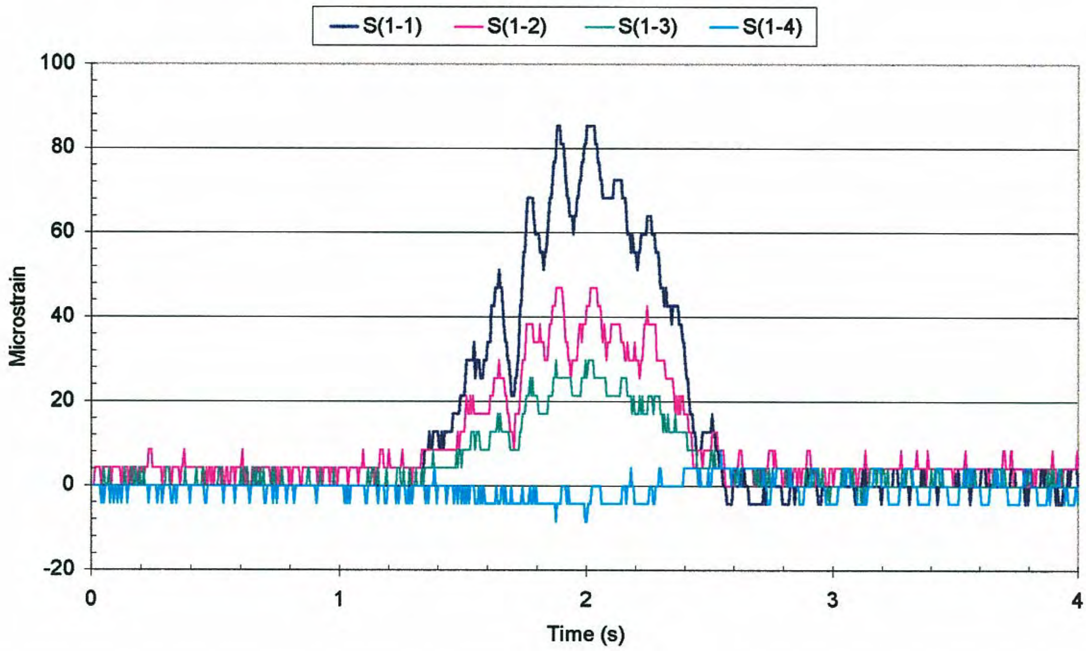
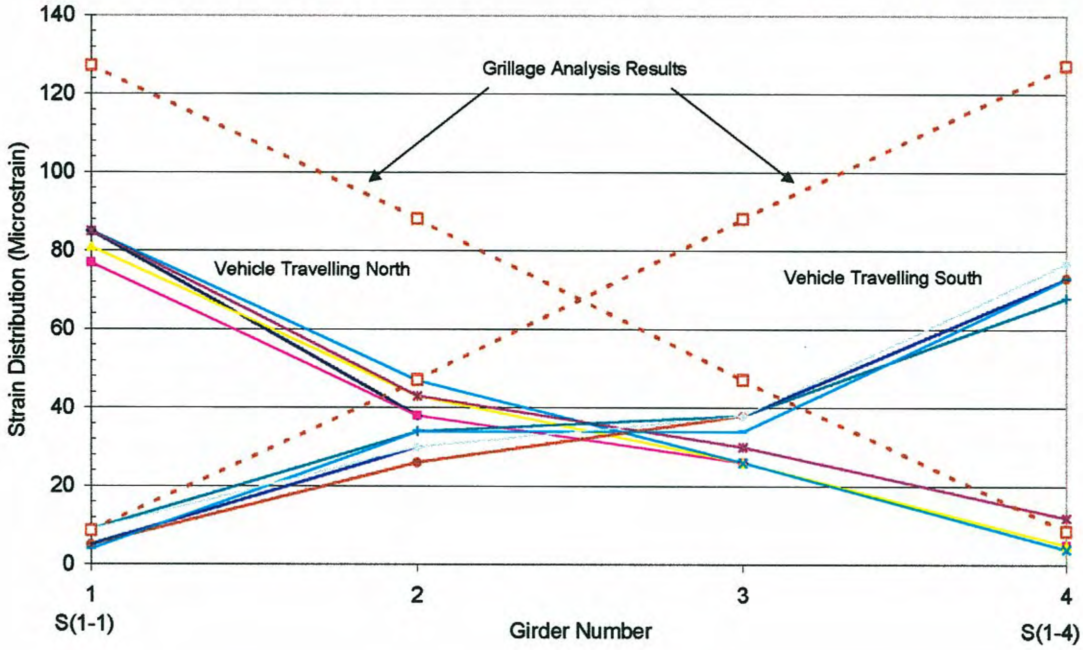



**Figure 4.17 Strain distribution for known vehicle testing at different speeds and in two directions.**



The distribution of strain into each of the girders from the known vehicle data is presented in Figure 4.17. The distribution presented is consistent with the data collected from health monitoring of the ambient heavy vehicle traffic, with higher strains recorded in the edge girders compared with those recorded in the middle girders. The distribution also shows that higher strains were recorded in Girder 1 for vehicles travelling to Waipara (north), and that, for vehicles travelling to Christchurch (south), the maximum strain was recorded in Girder 4 as expected.

Figure 4.17 also illustrates the results from the grillage analysis which 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 the guardrail. The vehicle position for the grillage analysis was 600 mm out from the kerb. The differences between the theoretical and recorded results could be related to variations in the vehicle position on the bridge, and possibly the influence of bearing restraint effects.

A single (0.85 HN) evaluation vehicle should induce a strain of 96 $\mu\varepsilon$, based on a grillage analysis. Given that the largest recorded strain resulting from the passage of the known vehicle was 85 $\mu\varepsilon$, the known vehicle induced effects in the bridge that were approximately 90% of the evaluation vehicle.

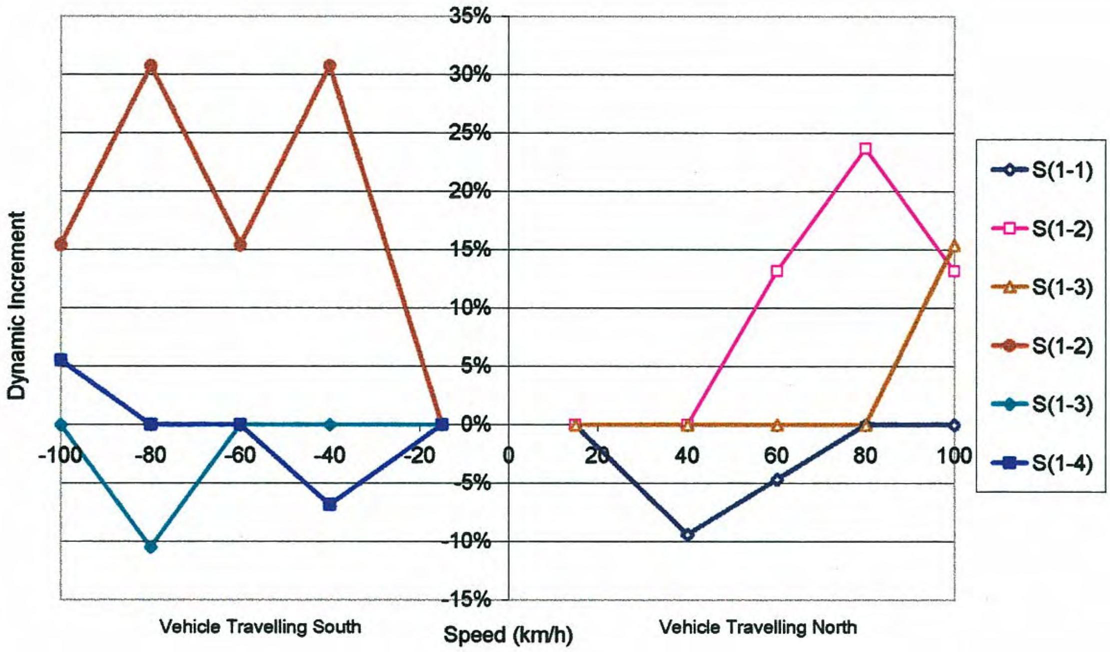
The free vibration response of the main girders in the structure after the vehicles had driven off the span was quite large. The impact factor or dynamic increment is used to indicate the increase in the effect of a vehicle on a structure as the speed increases. The dynamic increment (impact factor) (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 4.18. These results show a high dynamic response at 40 km/h and 80 km/h for the known vehicle travelling south. For the two directions of travel, only the three transducers most affected by the passage of the vehicle in that direction are presented in Figure 4.18. The maximum value of 30% was recorded at 80 km/h. This is a similar value to the value recommended by the Bridge Manual for this bridge.

The natural frequency of the structure was found to be approximately 7.4 Hz, based on the passage of the known vehicle. The level of damping in the superstructure is approximately 1.8%. The higher dynamic response for the vehicles travelling north may be related to the effects of the road profile adjacent to the abutment of the southern approach.

**Figure 4.18 Dynamic increment plot for known vehicle.**




## 4.5 Summary

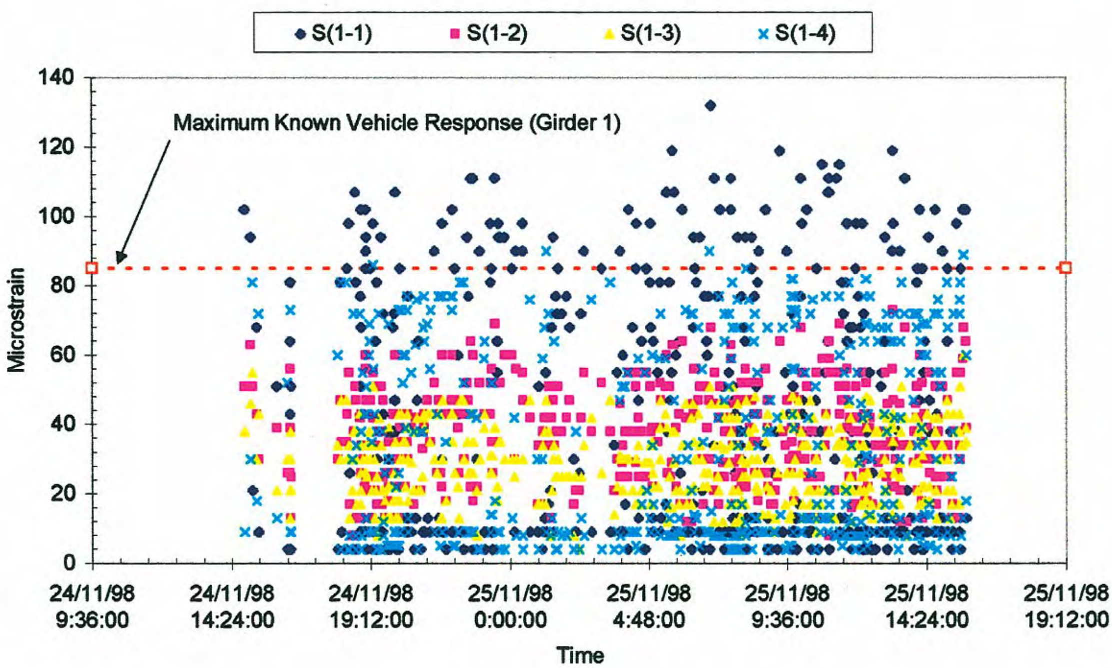
A summary of the data recorded for the Health Monitoring programme and the testing with the known vehicle is presented in Table 4.3. The maximum recorded midspan girder strains obtained from the Health Monitoring exceed the maximum strains recorded for the known vehicle by approximately 50%. The known vehicle induced bending moments on the span that are equivalent to 90% of the bending moments from the 0.85 HN evaluation load. The implication of these two observations is that some overloading may be occurring on this route, particularly in the northbound direction. The higher strains for the known vehicle travelling north may be related to the effects of the poor road profile of the southern approach.

A comparison of the maximum response to the known vehicle and the response of the bridge to the overall ambient heavy vehicle traffic is illustrated on the scatter diagram in Figure 4.19 for transducer S(1-1). This figure shows that the magnitude of effects resulting from heavy vehicles is significantly greater than from the known vehicle, and that a significant number of these events occur. In particular, it is unlikely that many, if any, of the strains recorded above the known vehicle line on Figure 4.19 are the result of multiple presence events.

**Table 4.3 Summary of health monitoring data.**

| Transducer | Maximum Recorded Value (Known Vehicle) | Maximum Recorded Value (Health Monitoring) | Extrapolated Value (95% confidence limit) for 100 years |
|---|---|---|---|
| | *Strain ($\mu\varepsilon$)* | | |
| S(1-1) | 85 | 130 | 205 |
| S(1-2) | 47 | 72 | 115 |
| S(1-3) | 38 | 55 | 80 |
| S(1-4) | 77 | 90 | 145 |
| S(CG-2) | 47 | 85 | 135 |
| | *Deflection (mm)* | | |
| D(1-1) | 2.3 | 4.2 | 6.7 |
| D(1-2) | 2.0 | 3.2 | 5.3 |
| D(1-3) | 2.0 | 2.7 | 3.4 |

**Figure 4.19 Comparison of maximum response of the bridge to the known vehicle with its response to the ambient heavy traffic.**



# 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 Waipara Bridge. However, because the theoretical analysis showed that this bridge has adequate strength, the objectives of this report were to evaluate the overall performance of the bridge and the traffic effects on this bridge. This evaluation was conducted principally by measuring midspan effects and a Fitness for Purpose Evaluation of the superstructure was made, based on midspan bending.

## 5.2 Girder Bending

### 5.2.1 Multiple Presence

The Waipara 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 diagram shows a transverse distribution of strain for each direction and the sum of these two distributions for both the 95% in 1 year (serviceability) event and the 95% in 100 year (ultimate) event. The data show that the highest strain caused by a multiple presence event occurs in Girder 1. The maximum strains include 215$\mu\varepsilon$ (95% in 1 year) and 240$\mu\varepsilon$ (95% in 100 years).

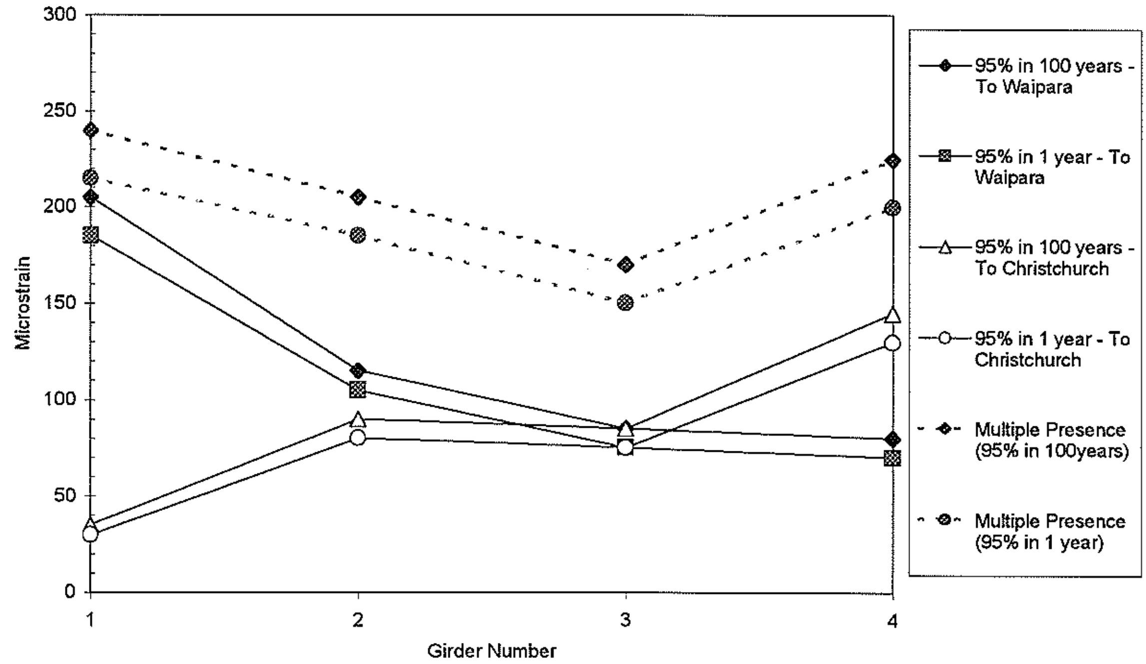



**Figure 5.1 Multiple presence effects using the approach that is consistent with the Bridge Manual.**

### 5.2.2 Moment versus Strain Relationship

Figure 5.2 illustrates a theoretical moment versus strain curve for a typical girder of the Waipara 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 cracks, 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. In this case the service load capacity relates to the load at which cracking occurs in the soffit of the beam.

Figure 5.2 also presents the reduced capacity ($\phi M_u$) of a typical girder converted to an equivalent strain, based on the theoretical moment versus strain relationship. The dead load and factored dead load moment, and corresponding strains, are presented and a shaded rectangle represents the multiple presence effect under service loads.

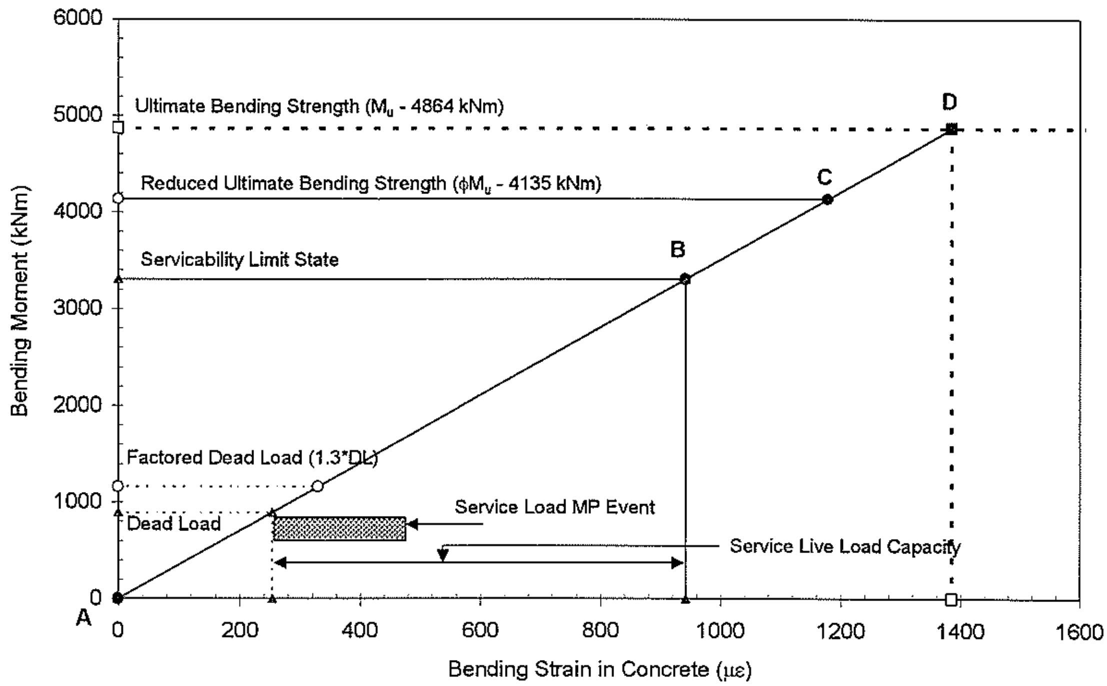



**Figure 5.2 Moment versus strain relationship, and summary of Fitness for Purpose for Girder 1.**

### 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 1 are listed in Tables 5.1 and 5.2. These two evaluations show that the structure has significant reserves of strength in bending.

**Table 5.1 Summary of Fitness for Purpose Evaluation based on the Ultimate Bending Capacity of Girder 1.**

| Item | Result |
|---|---|
| Ultimate Strength ($\phi M_u$) | 4864 kNm |
| Dead Load (*1.3) | 1157 kNm |
| Ultimate Live Load Capacity Moment – Ultimate ($\gamma_o R_o$) | 3707 kNm |
| Ultimate Live Load Capacity – Equivalent Strain ($\gamma_o R_o$) | 1055 $\mu\varepsilon$ |
| Ultimate Traffic Load Effect (Multiple Presence) | 240 $\mu\varepsilon$ |
| Fitness for Purpose Evaluation | 440% |

**Table 5.2 Summary of Fitness for Purpose Evaluation based on the Serviceability Bending Capacity of Girder 1.**

| Item | Result |
|---|---|
| Serviceability Strength ($M_o$) | 3305 kNm |
| Dead Load | 890 kNm |
| Ultimate Live Load Capacity Moment – Serviceability ($\gamma_o R_o$) | 2415 kNm |
| Serviceability Live Load Capacity – Equivalent Strain ($\gamma_o R_o$) | 690 $\mu\varepsilon$ |
| Serviceability Traffic Load Effect – 95% in 1 year (Multiple Presence) | 215 $\mu\varepsilon$ |
| Fitness for Purpose Evaluation | 320% |

## 5.3 Summary

The Fitness for Purpose Evaluation calculated for Waipara Bridge is based on the midspan bending of the main girders. For the serviceability limit state it is 320%, and for the ultimate limit state it is 440%. These evaluations are significantly higher than the theoretical evaluations, which for the serviceability limit state is 235%, and for the ultimate limit state is 155%. Comparison with the posting evaluations is the most relevant to Health Monitoring as these evaluations are based on loads which are more representative of ambient traffic.

The Fitness for Purpose Evaluation based on midspan bending suggests that the performance of the Waipara Bridge is approximately twice as good as theoretical predictions. While this appears to be a very significant increase in its estimated capacity, it is justifiable. Factors contributing to this improvement are potentially seizure of the main girder support bearings, favourable lateral position of the vehicles, and reduced dynamic effects in some bridge elements.

This apparent performance improvement based on midspan bending cannot be utilised, since web shear (not midspan bending) governs girder capacity. This behaviour was not monitored during the Health Monitoring programme. Two significant points follow from this observation:

1. Identifying and monitoring the critical failure modes are necessary to gain maximum benefit from Health Monitoring.
2. Even if web shear behaviour had been monitored, insufficient research has been conducted to understand how the web shear capacity of the girders is modified by the parameters that have improved the girder bending behaviour.

# 6. Conclusions

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

*Theoretical Analysis*
A theoretical analysis of the bridge found that the web shear capacity of the main girders is the critical issue with this bridge and that the 0.85 HO rating evaluation based on this failure mode is 110%.

The capacity of the bridge was also assessed for both the ultimate and serviceability limit states for midspan bending. However, as this bridge has sufficient capacity to withstand the evaluation loads, the Health Monitoring programme focused on evaluating the overall performance of the bridge and the effect on the bridge of the heavy vehicle traffic using this route.

*Health Monitoring Results*

- The ambient heavy vehicle traffic is inducing bending moments in the bridge that are 50% higher than the known vehicle. Given that the known vehicle produced effects of a magnitude 90% of the 0.85 HN-evaluation vehicle, this indicates that significant overloading occurs on this route, particularly with the northbound traffic.
- The recorded strains and deflections at midspan are significantly lower than the theoretically predicted strains. This is probably related to variations in vehicle position on the bridge, and bearing restraint effects.
- The uneven road profile on the southern approach to the bridge is also expected to increase the effect of vehicles on the bridge.

*Fitness for Purpose Evaluation*

- The Fitness for Purpose Evaluation, based on midspan bending, for the serviceability limit state of this bridge is 320%, and for the ultimate limit state is 440%. These evaluations are significantly higher than the theoretical evaluations. The higher Fitness for Purpose Evaluations are expected to relate mainly to bearing restraint effects, the influence of the lateral position of vehicles on the bridge, and to bridge and vehicle dynamic effects.
- The theoretical evaluation for the posting load, based on midspan bending, for the serviceability limit state is 235%, and for the ultimate limit state is 155 %.
- The posting evaluations are the most relevant to apply to Health Monitoring as they are based on loads which are more representative of actual traffic.

- The Fitness for Purpose Evaluation based on midspan bending cannot be used directly as a means of bridge assessment.
- Girder web shear governs the capacity of the Waipara Bridge, and this was not monitored during the Health Monitoring programme. In addition there is only limited understanding of girder web shear capacity, when factors such as bearing restraint become significant.

# 7. Recommendations

The recommendations are that:

- Web shear effects should be measured in addition to midspan bending effects when health monitoring this type of prestressed concrete bridge.
- Issues associated with the bearing behaviour on this particular bridge should also be investigated.
- More research is required concerning shear capacity of concrete members, when factors such as bearing restraints are significantly influencing member behaviour.

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