Health Monitoring of Superstructures of New Zealand Road Bridges:

Otauru Bridge, Manawatu

Transfund New Zealand Research Report No.169

Health Monitoring of Superstructures of New Zealand Road Bridges:

Otauru Bridge, Manawatu

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. 169

ISBN 0-478-11577-6
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: Otauru Bridge, Manawatu. *Transfund New Zealand Research Report No. 169*. 46pp.

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, Palmerston North, 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 Otauru Bridge, on State Highway 57, crosses the Otauru Stream between Levin and Pamerston North, Manawatu Region, North Island, and is essentially a large reinforced concrete culvert. It was selected as one of these ten because, during inspection of the structure, it had a large crack at midspan, and the structure was visibly deflecting under traffic.

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

Theoretical Analysis

The theoretical assessment of the superstructure of the bridge found that the 0.85 HO* rating evaluation was 75%, and the 0.85 HN* posting evaluation was 95%. This compares poorly with the value obtained from the Transit New Zealand Structural Inventory of 95% for the 0.85 HO rating evaluation, but this difference may be explained by variations in the assumptions used during the rating analysis. While the bridge should technically be posted with a load limit, the need for this based on theoretical analysis is marginal.

Health Monitoring Results

The findings from the Health Monitoring are that:

- High strains were recorded over the transverse crack at midspan indicating that a plastic hinge may have formed at midspan and that the structure may not be acting as originally designed.
- The level of overloading on this route is well controlled.
- The dynamic increment or impact factor recorded for vehicles travelling towards Palmerston North is around 50%. This is much higher than the value recommended in the Transit New Zealand (1994) Bridge Manual of 30% for this structure. This high dynamic increment is caused by the poor road profile on the bridge approach.

* HO Highway overweight vehicles; HN Highway normal vehicles

Fitness for Purpose Evaluation

- The Fitness for Purpose Evaluation for this bridge, based on the critical midspan bending of the slab, was 30%, which indicates that intervention is required.
- The structure is thought to be acting as a three-pinned arch, rather than as a portal frame on which the Fitness for Purpose Evaluation was based.
- The high strains at midspan may lead to a fatigue failure of the plastic hinge.

Recommendations

The recommendations are that:

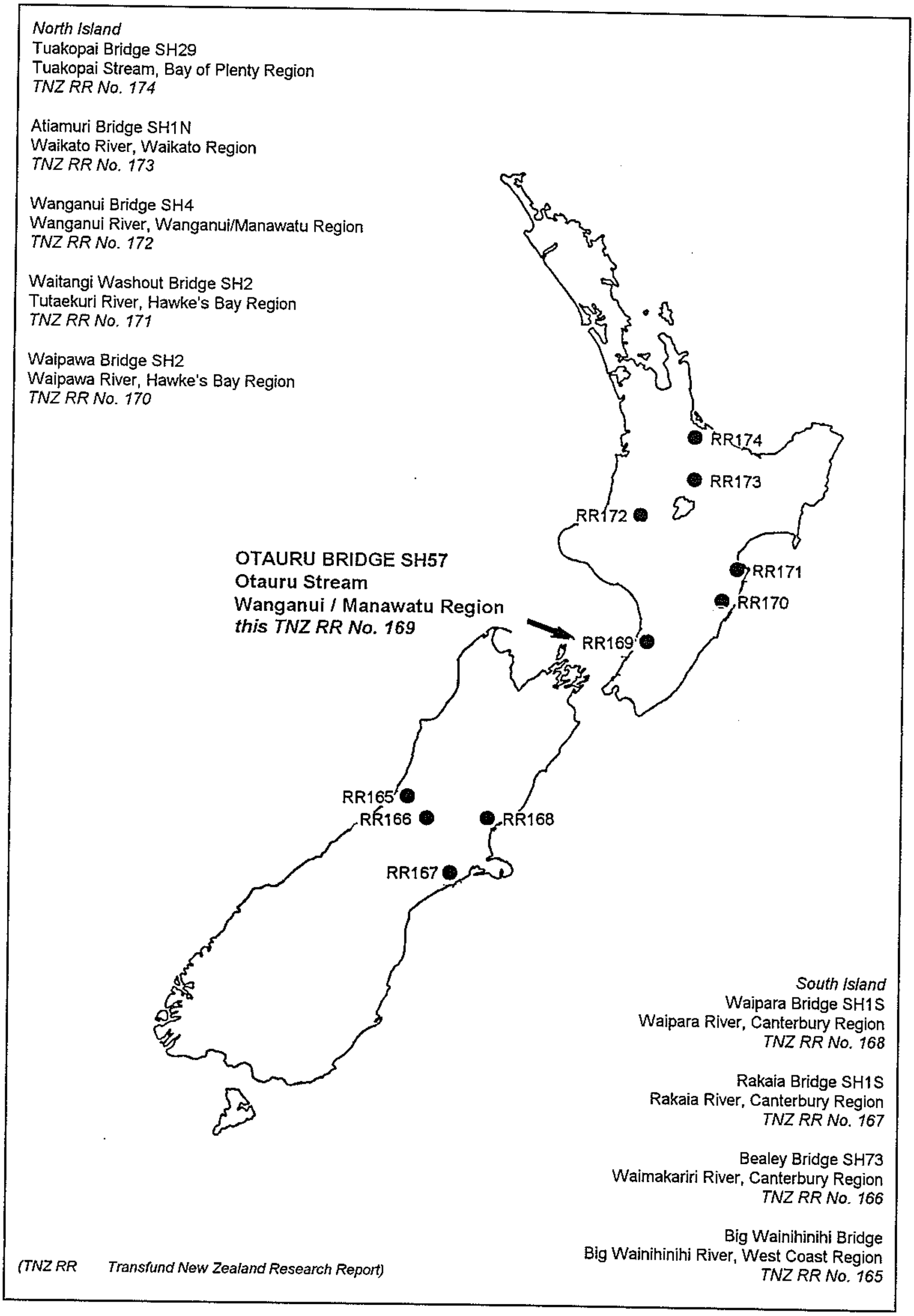
- This structure requires remedial action.
- The Otauru Bridge should be further investigated
  - to determine if its current condition and performance is acceptable, and
  - to design and implement remedial action.
- Improvement of the road profile should be considered to reduce the effects of vehicles on this structure.

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 Otauru Bridge, on State Highway 57, crosses the Otauru Stream between Levin and Palmerston North, Manawatu Region, North Island, and is essentially a large reinforced concrete culvert. It was selected as one of these ten because it has a large crack at midspan, and the structure was visibly deflecting under traffic. The Fitness for Purpose Evaluation indicates that intervention is required.

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



1. Introduction

1.1 Bridge Health Monitoring

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

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

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

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 Otauru Bridge, on State Highway (SH) 57, which crosses the Otauru Stream between Levin and Palmerston North, Manawatu Region, North Island of New Zealand (Figure 1.1). The reasons for choosing this bridge for the representative sample were:

- It is essentially a large reinforced concrete culvert;
- It had a large crack at midspan; and
- The structure was visibly deflecting under traffic.

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

1.2 Applying Health Monitoring Technology

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

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

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

- Midspan bending strength of the concrete slab.
- Shear strength of the concrete slab.

The substructure was not considered 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.

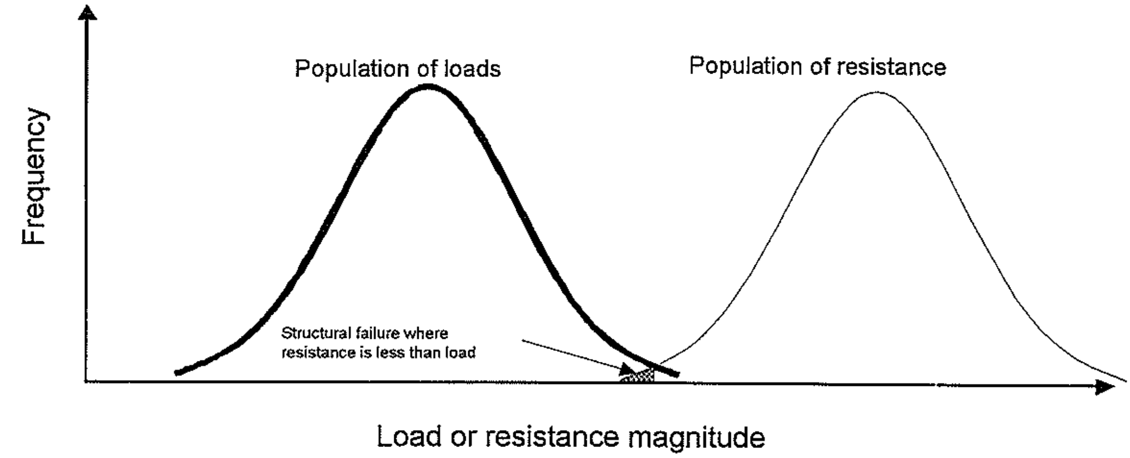


Figure 2.1 Statistical representation of structural failure.

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 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.

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

where:

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

2.3.1.1 *Rating Evaluations*

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

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

2.3.1.2 *Posting Evaluations*

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

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

2.3.2 Decks

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

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

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

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

2.4 The Health Monitoring Approach

2.4.1 Theory of this Approach

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

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

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

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

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

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

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

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

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

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

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

where:

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

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

2.4.2 Behavioural Test of this Approach

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 Otauru Bridge during the Health Monitoring programme, and the results are given in section 4.4 of this report.

3. Bridge Description & Assessment

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

3.1 Bridge Description

The Otauru Bridge, located on State Highway (SH)57, crosses the Otauru Stream between Levin and Palmerston North, Manawatu Region. The structure is essentially a large culvert constructed from reinforced concrete and consists of a single span including a concrete slab that is framed into the abutments. The structure is designed to act as a portal frame with a span length of 10.4 m and carries two lanes of traffic. Construction of the bridge was completed in 1941. The bridge is illustrated in Figure 3.1. It has a large transverse crack across the concrete slab at midspan and the structure moves visibly under load. This crack may be caused by restrained shrinkage or is live load-induced.

Figure 3.1 Otauru Bridge on SH57 (looking north).

The Otauru Bridge has a current theoretical load rating of the bridge listed in the TNZ Structural Inventory (1999) of:

- Bridge Class (superstructure) 95%
- 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.

The theoretical behaviour of the bridge was investigated using a frame analysis computer package. The dimensions of the structure used in the analysis were taken from the "as constructed" plans, and were confirmed by on-site measurements. The structural analysis assumed that the structure acted as a frame with the concrete slab framed into the abutments. The legs of the frame were modelled with pins at the base, and an elevation of the model used is shown in Figure 3.2. The section properties considered the effect of the kerbs but not the guardrails.

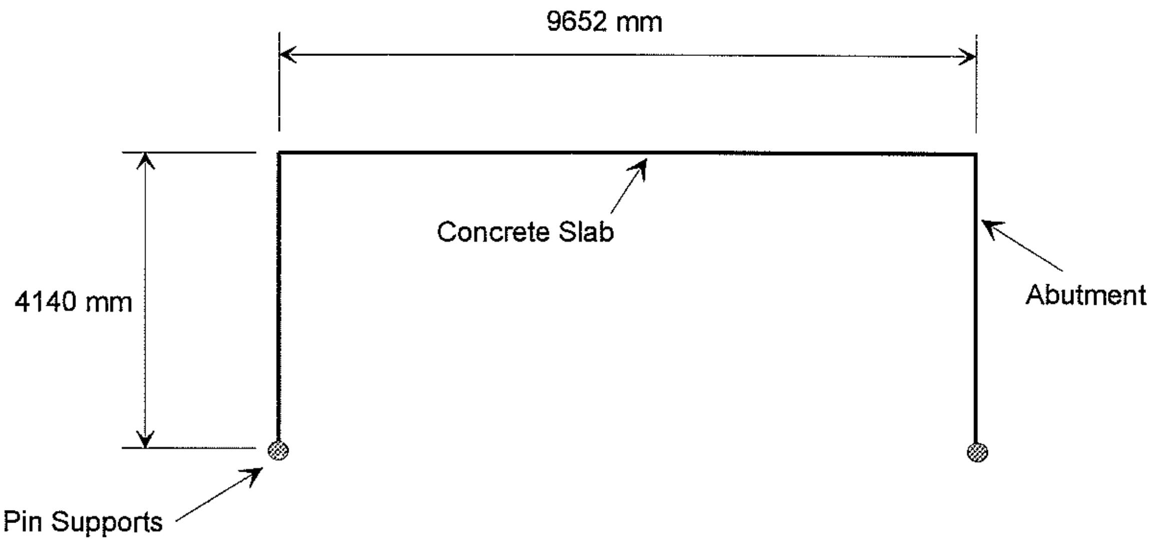


Figure 3.2 Representation of model used.

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

- Concrete $f'_c = 21$ MPa, $E = 22\,100$ MPa
- Steel Reinforcement $f_y = 230$ MPa, $E = 200\,000$ MPa

3.2.1 Slab Bending

The maximum bending moment at the midspan of the slab caused by the dead load is 40 kNm per metre width of slab. The maximum fixed end bending moment caused by dead load at the intersection of the slab and the abutments is approximately 80 kNm per metre width of slab. A summary of the maximum bending moments resulting from the various loads applied to the grillage model is presented in Table 3.1. The results in the table are not factored and represent the extreme bending moment in a one-metre width of the slab with the vehicle at the greatest allowable eccentricity. Bending moments are presented for midspan bending of the slab and for the negative moment in the slab at the abutment.

Table 3.1 Results of structural analysis for bending moments of the slab.

Load	Abutment Bending Moment (kNm/m)	Midspan Bending Moment (kNm/m)
Dead Load	–80	40
Known Vehicle	–36	16
2x 0.85 HN Vehicles (Posting Load)	–45	25
0.85 HO + 0.85 HN Vehicles (Rating Load)	–75	40

The bending capacity of the concrete section of the superstructure was calculated in accordance with Section 8 of the Concrete Structures Standard (NZS 3101: Part1 1995). The midspan bending capacity (ϕM) is 110 kNm/m and the bending capacity at the abutment is 272 kNm/m.

3.2.2 Shear Strength

The maximum shear force in a one-metre width of the slab determined from the grillage analysis is presented in Table 3.2.

Table 3.2 Results of structural analysis results for shear in the girders.

Load	Shear Force (kN/m)
Dead Load	50
Known Vehicle	25
2x 0.85 HN Vehicle (Posting Load)	30
0.85 HO + 0.85 0.85 HN Vehicles (Rating Load)	50

The shear capacity of the deck slab was calculated in accordance with Section 9 of the Concrete Structures Standard (NZS 3101: Part 1 1995). The shear capacity (ϕV) is 200 kN/m.

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. The rating and posting evaluations have been assessed for the bending and shear in the slab only. The table also presents a comparison of the load evaluations calculated by Infratech Systems & Services (Infratech), and the load rating recorded in the 1999 TNZ Structural Inventory. A value of 1.3 was used for the impact factor in calculating the load evaluations, and a value of 1.25 was used for the dead load factor.

Table 3.3 Summary of theoretical load evaluations for the superstructure.

Mode of Failure	ϕ Ultimate Capacity	0.85 HO Rating Load	0.85 HN Posting Load	Dead Load	0.85 HO Rating (Infratech	0.85 HN Posting (Infratech)	Rating (Structural Inventory)
Abutment Bending	–275kNm/m	–75kNm/m	–45kNm/m	–80kNm/m	120%	155%	95%
Midspan Bending	110kNm/m	40kNm/m	25kNm/m	40kNm/m	75%	95%	
Girder Shear	200kN	50kN	30kN	50kN	140%	180%	

The overall rating of the superstructure is taken as the minimum rating of all the components. For this bridge, the load rating (75%) is the minimum of the ratings based on shear and bending, and the critical failure mode is midspan bending of the deck slab. This evaluation compares poorly with the rating of 95% which is documented in the TNZ Structural Inventory. The differences may be associated with assumptions regarding material properties and/or structural actions.

Based on the calculations summarised in Table 3.3, the capacity of the structure is considered marginal with respect to posting requirements based on midspan bending. The superstructure is indeterminate with large reserves of strength at the abutments. Thus at the strength limit state, some moments can be re-distributed from the midspan to the supports.

3.4 Summary

The Otauru Bridge, in Manawatu Region, was analysed using a frame analysis computer package to determine the bending moment and shear in the slab, based on various vehicle loadings. The midspan bending moment of the slab was found to govern the strength and therefore determines the rating of the superstructure.

Based on the results from this analysis, the Health Monitoring programme concentrated on determining a Fitness for Purpose for the slab based on midspan bending.

4. Health Monitoring Programme

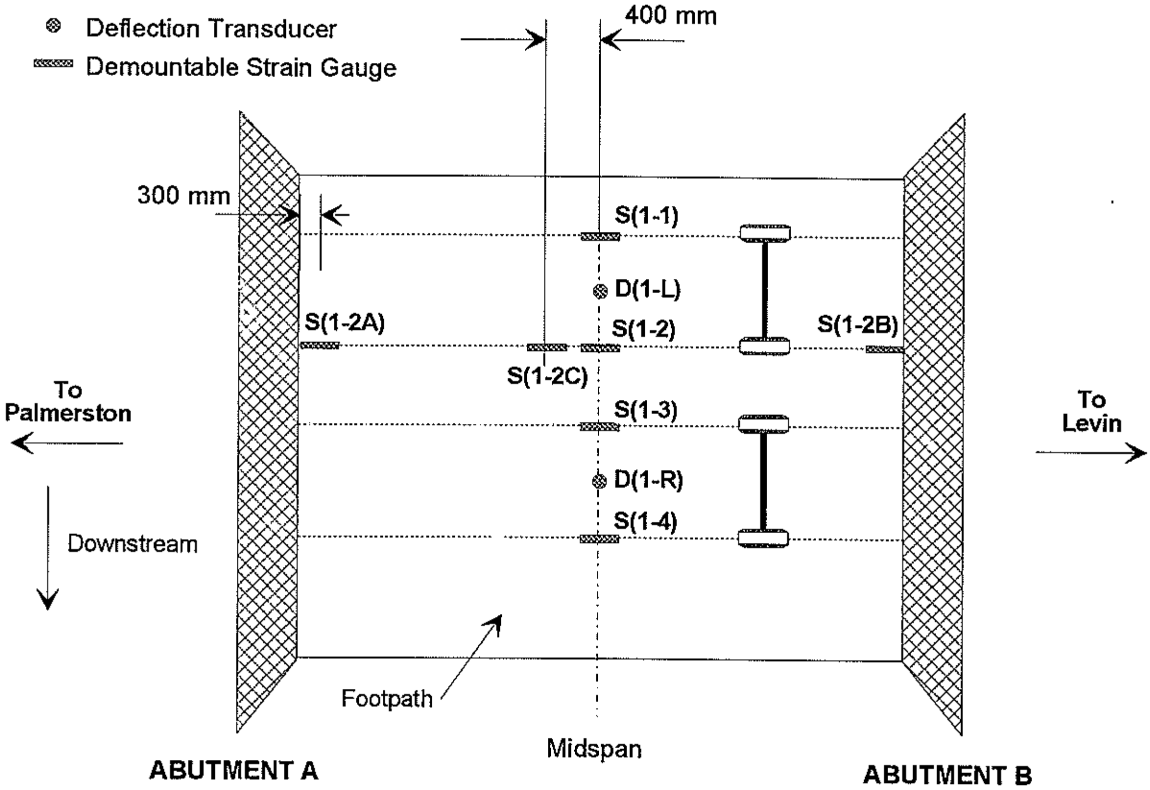
The programme of Health Monitoring on the Otauru Bridge involved two components:

- 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 Otauru Bridge.

4.1 Instrumentation

The instrumentation installed on the bridge included six Demountable Strain Gauge transducers and two Deflection transducers. The locations of this instrumentation are illustrated in Figure 4.1, which also shows the approximate wheel paths of the vehicle on the bridge. These wheel paths are illustrated on Figure 4.2, a cross section of the structure. The transducers were positioned directly under the wheel paths in order to record the maximum possible strains in the soffit of the slab.



S = demountable strain gauge; D = deflection transducer

Figure 4.1 Instrumentation plan for Otauru Bridge.

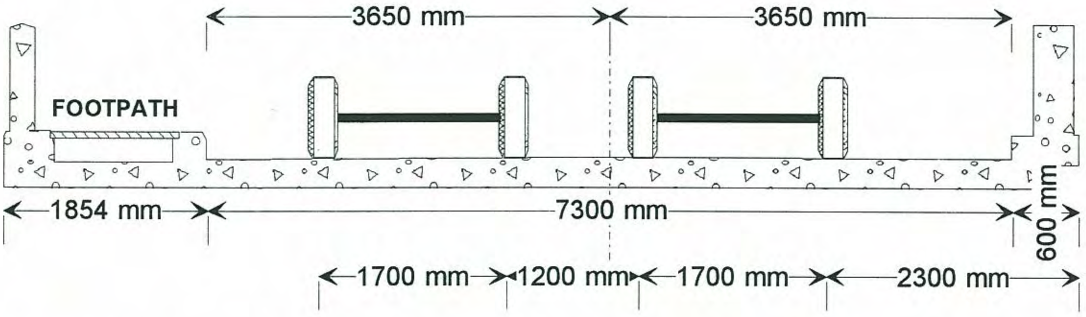


Figure 4.2 Cross section (looking north) of the bridge structure showing lateral position of vehicles in each lane.

Figure 4.1 also shows a ninth transducer that was used during the testing. At approximately 11am on 9 October, 1998, transducer S(1-2A) was removed from its position and installed adjacent to transducer S(1-2). The new name for this transducer was S(1-2C).

The four transducers installed across the midspan of the bridge were all positioned over a large crack in the soffit of the deck. Transducer S(1-2C) was placed adjacent to S(1-2) but away from any cracking in order to compare the difference in strain at these two points. Figure 4.3 illustrates the position of a demountable strain gauge over a typical crack in the slab.

Figure 4.3 Demountable strain gauge over a crack in the slab.

Transducers S(1-2A) and S(1-2B) were installed close to the fixed supports (abutments) of the structure to determine the degree of negative bending at these locations. These could be compared to the midspan bending strains and the analytical analysis to determine the behaviour of the structure.

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

Figure 4.4 Instrumentation installed at the midspan of the slab.

4.2 Procedure

The health monitoring of the structure began on Friday 9 October, and continued until Saturday 10 October 1998, giving a total monitoring period of approximately 22 hours. During the one-day monitoring period, the response of the bridge to 311 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 Friday 9th October, 1998. The known vehicle was a heavy vehicle of known mass and dimensions (illustrated in Figure 4.5), with a gross mass of approximately 46 tonnes, and was supplied by Renall Haulage, Palmerston North. It is shown on the bridge in Figure 4.6.

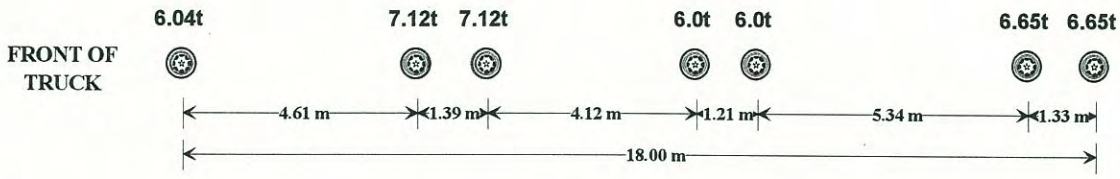


Figure 4.5 Axle mass and configuration of the known vehicle.

Figure 4.6 The known vehicle on the Otauru Bridge during the behavioural testing.

The testing with the known vehicle was conducted by recording the response of the bridge to the vehicle as it passed over the bridge at different speeds. The tests were conducted with the vehicle travelling in both directions (to Palmerston North and to Levin) at speeds ranging between a crawl (20 km/h) and 70 km/h, in increments of 10 km/h.

The lateral position of the known vehicle was in the normal lane, as shown in Figure 4.6. Testing was completed by slowing the traffic in each direction or in some cases stopping it for a few minutes at a time. This ensured minimal traffic interruptions and also allowed the continuous monitoring of ambient heavy vehicles between test runs with the known vehicle.

4.3 Short-Term Health Monitoring Results

4.3.1 Structure Response

Typical strain response versus time graphed (as waveforms) for the midspan bending strains recorded during the health monitoring for the passage of a heavy vehicle are presented in Figure 4.7. The waveforms represent a heavy vehicle travelling towards Levin.

Figure 4.7 Strain response versus time for midspan strain transducers for event recorded at 8.47am, 9 October 1998 (vehicle travelling towards Levin).

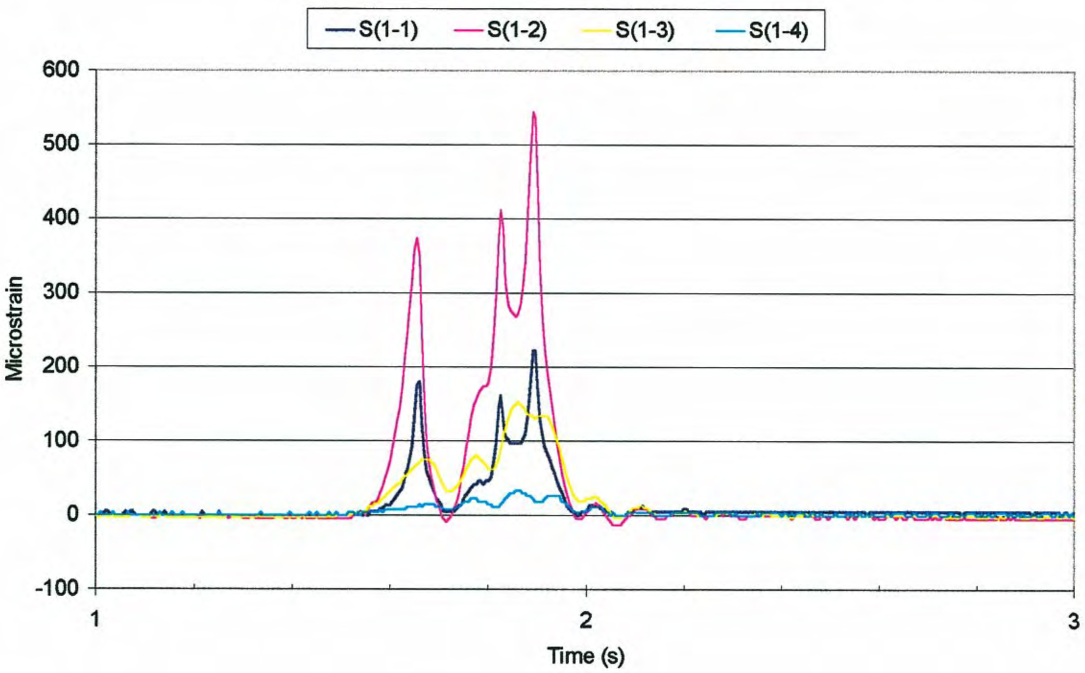
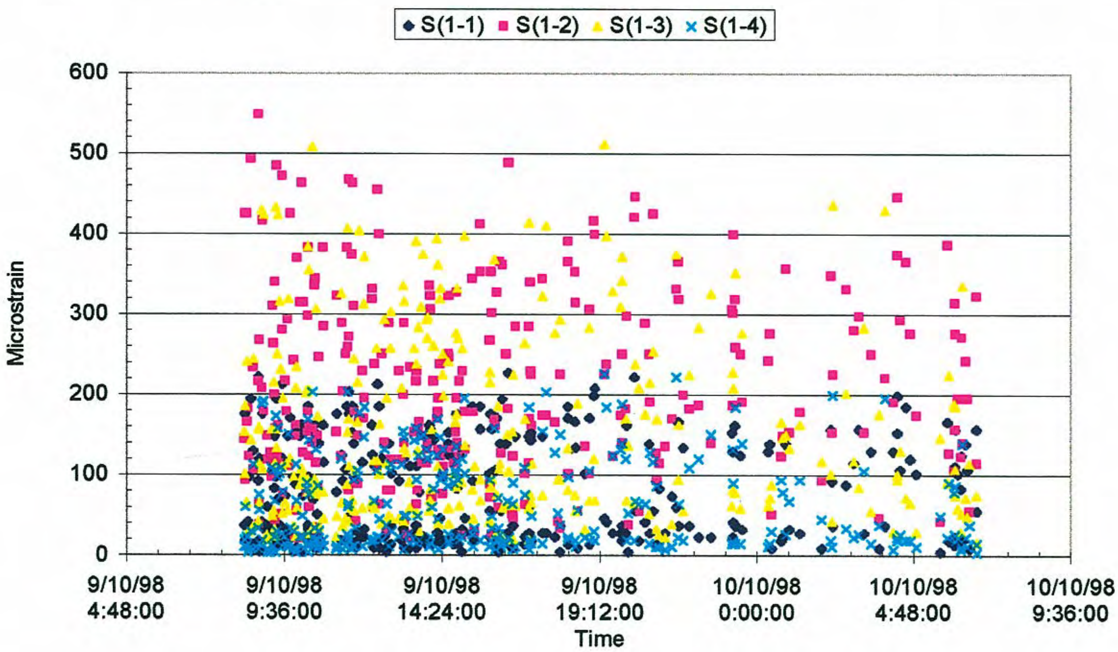


Figure 4.8 Scatter diagram for midspan strain transducers records for entire monitoring period.



A scatter diagram represents the maximum strains recorded during the passage of each heavy vehicle for the entire health monitoring period. Figure 4.8 presents the scatter diagram for the midspan transducers. 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 results show the difference in heavy vehicle traffic volumes between Friday (9/10/98) and Saturday (10/10/98).

The scatter diagram indicates that the greatest response in the soffit of the slab was recorded at the position of transducer S(1-2). The response of transducer S(1-3) was also quite large compared with the outside transducers S(1-1) and S(1-4). Generally the recorded strains are large in magnitude for a reinforced concrete structure.

The waveforms for the transducers positioned close to the abutments are illustrated in Figure 4.9. As expected, the recorded strains are negative, indicating that the fixed ends are causing negative moments at the position of these transducers. A scatter diagram for these transducers is presented in Figure 4.10. The magnitude of these strains is much smaller than the strains recorded at midspan.

The scatter diagram in Figure 4.10 illustrates only a small number of points for transducer S(1-2B), because this transducer was moved to a new position, adjacent to transducer S(1-2), and renamed S(1-2C) (see section 4.1).

Typical waveforms for the two midspan deflection transducers are presented in Figure 4.11. The waveforms illustrate a very similar response for the two transducers despite the different magnitudes. The scatter diagram for the deflection transducers is presented in Figure 4.12. Again the magnitude of these deflections is large for a reinforced concrete structure of this span.

A typical waveform for transducer S(1-2C) is presented in Figure 4.13. The transducer clearly shows the response to each individual axle load as the load passes over the transducer. The scatter diagram for this transducer is presented in Figure 4.14. While the response of this transducer is lower than those of the midspan transducers across the cracks, it is still high for this type of structure.

Figure 4.9 Strain response versus time for transducers installed adjacent to the abutments for event recorded at 8.47am, 9 October 1998 (vehicle travelling towards Levin).

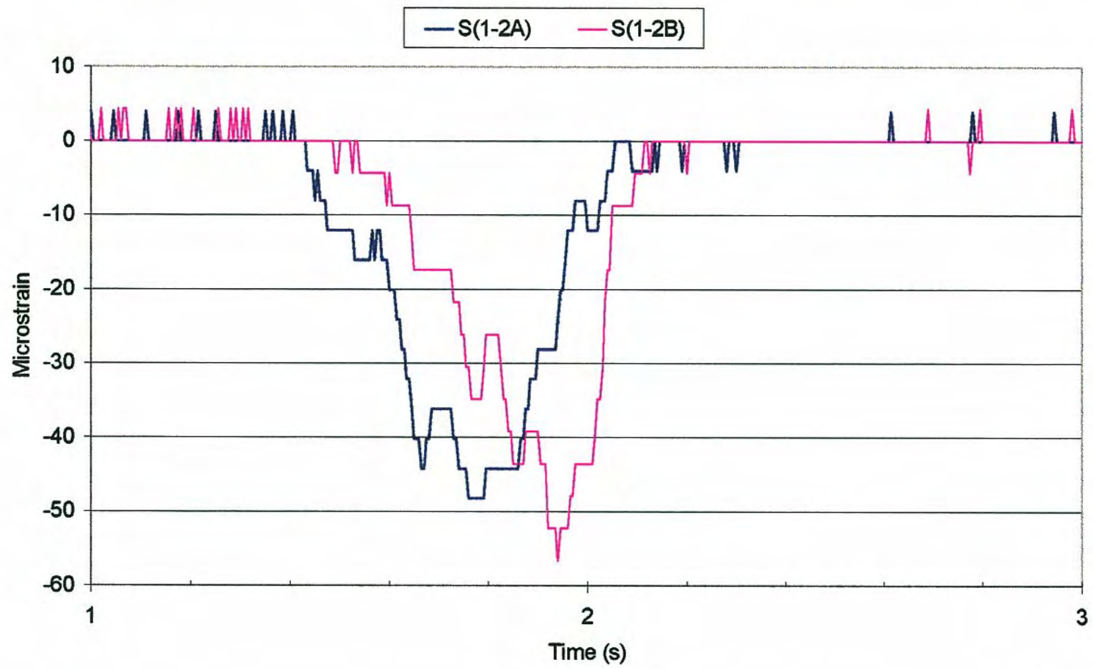


Figure 4.10 Scatter diagram for transducers installed adjacent to abutments.

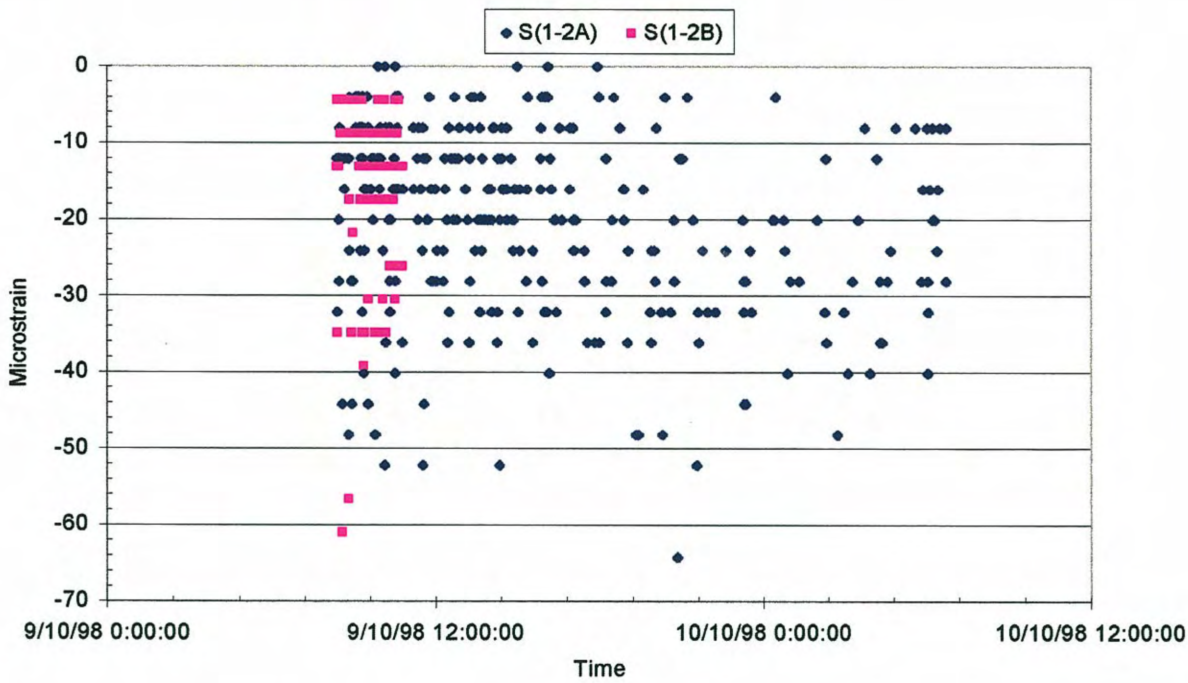


Figure 4.13 Strain response versus time for transducer S(1-2C) for event recorded at 3.54am, 10 October (vehicle heading towards Levin).

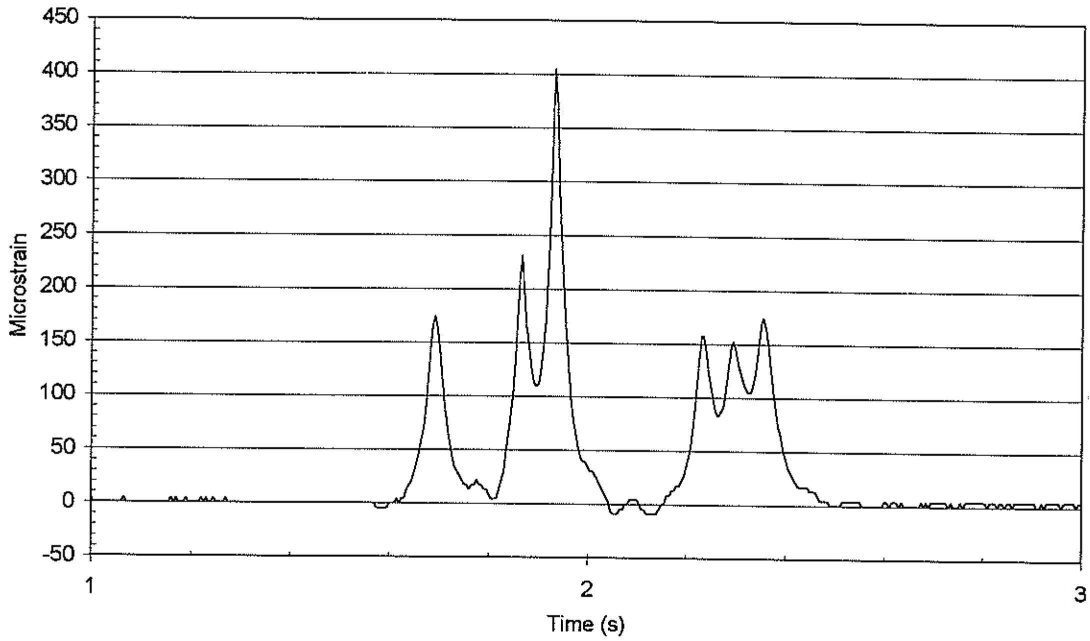
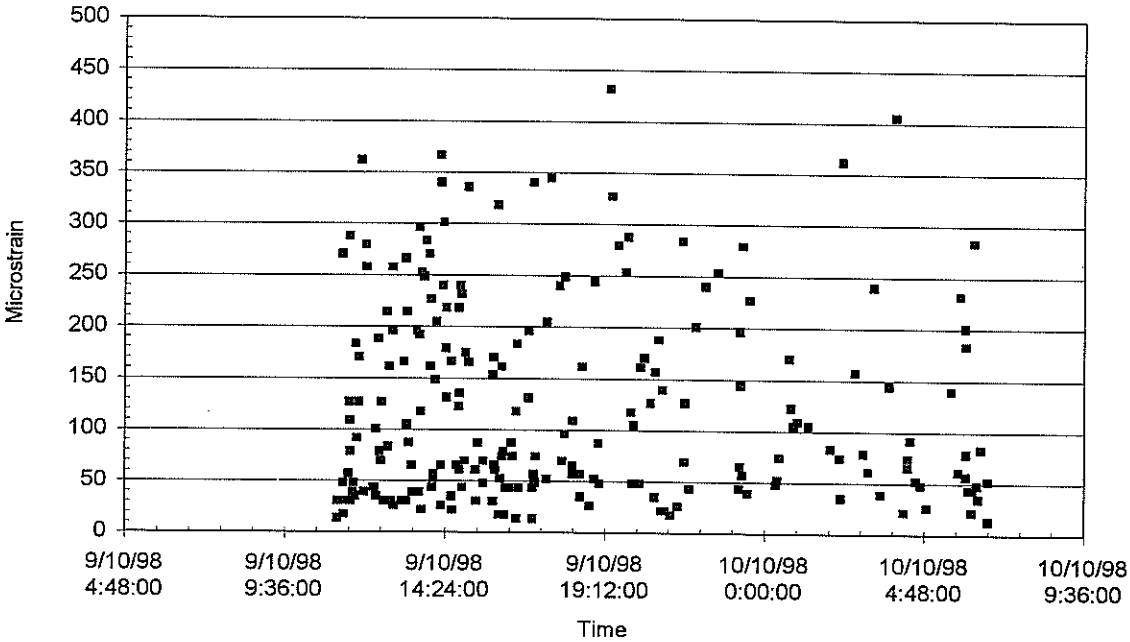


Figure 4.14 Scatter diagram for transducer S(1-2C) for its entire monitoring period.



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(1-2) in Figure 4.15.

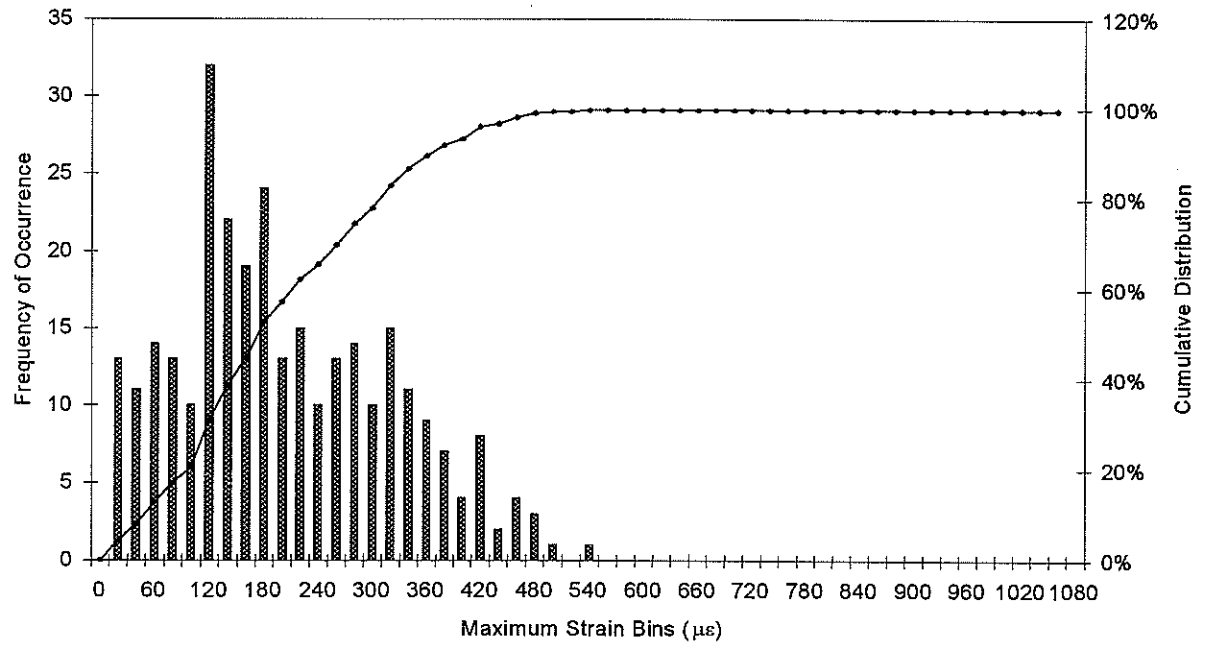


Figure 4.15 Histogram and cumulative distribution function for transducer S(1-2).

The cumulative distribution function can then be plotted on a probability scale known as an "inverse normal scale". The inverse normal plot for each of the strain transducers installed at midspan is presented in Figure 4.16. In this figure each curve has been plotted using only the data from the direction of vehicle travel that is causing the maximum response to each transducer.

On this graph the vertical scale represents the number of standard deviations that each point is away from the mean. The horizontal scale is the maximum strain recorded for each event. The point at which a data plot crosses the horizontal axis represents the average (mean) strain. A straight line represents a normally distributed sample of data.

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

The data in Figure 4.16 show significant differences in strain between transducers S(1-2) and S(1-3) located close to the centreline of the bridge compared with transducers S(1-1) and S(1-4) located towards the edge of the structure. The kerbs and guardrails are expected to significantly influence the behaviour of this structure. The additional stiffness of the guardrails may be reducing the strain in edges of the structure.

Figure 4.16 Inverse normal plots for strain transducers installed at the midspan.

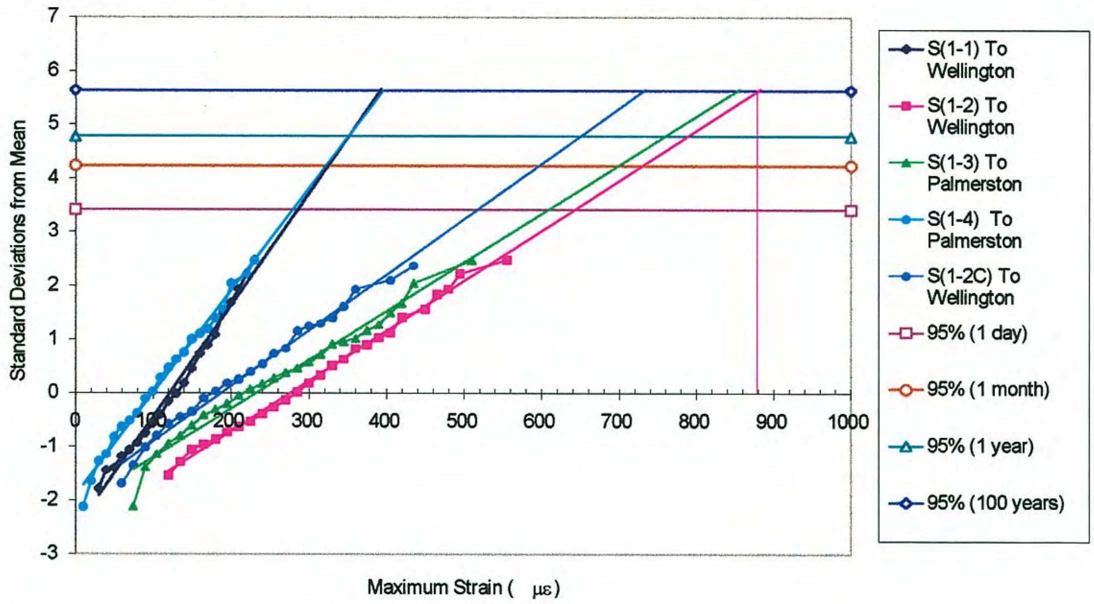


Figure 4.17 Inverse normal plots for transducers installed adjacent to the abutments.

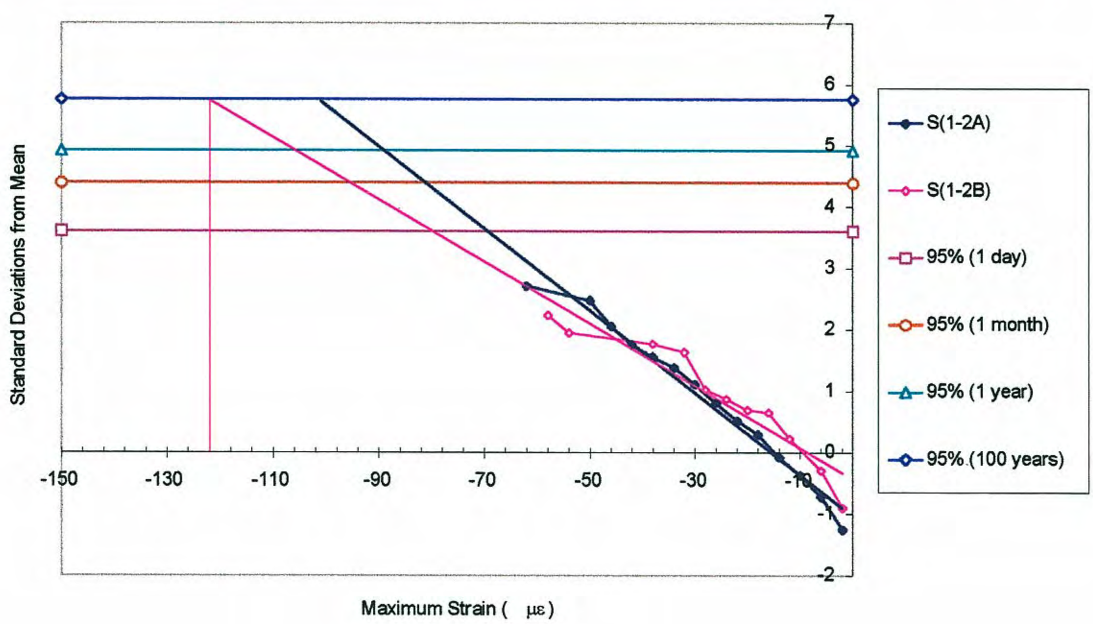
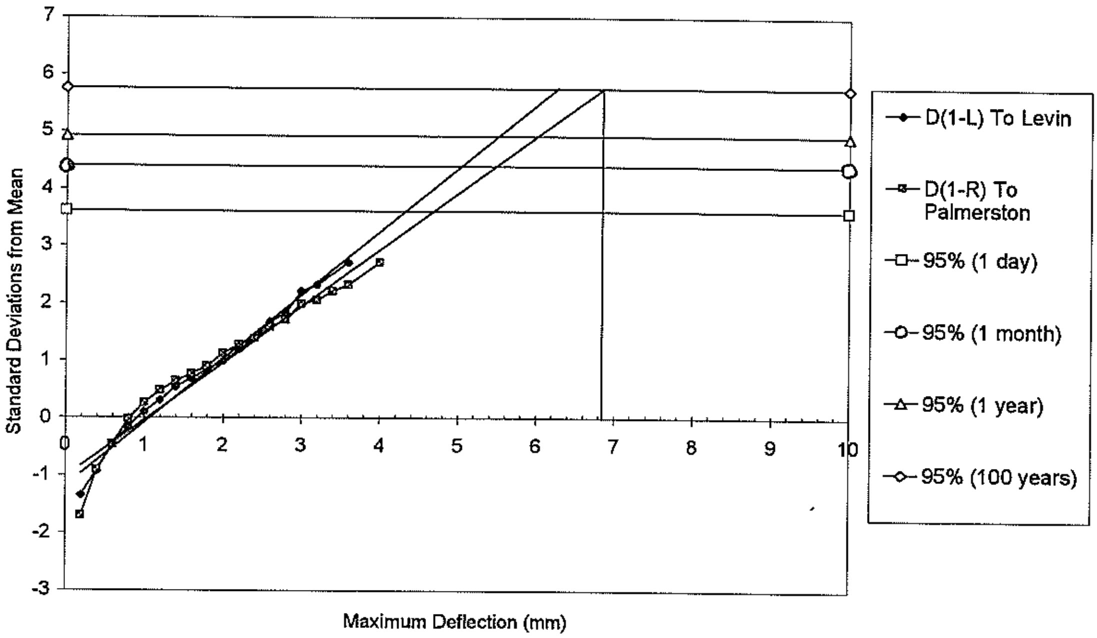


Figure 4.18 Inverse normal plots for deflection transducers.



The inverse normal plot shows that the strain extrapolated for the 95% confidence limit for 100 years (ultimate traffic load effect) is the highest for the midspan transducer S(1-2). The extrapolated value is approximately 880 µε.

The inverse normal plots for the transducers adjacent to the abutments are presented in Figure 4.17 for the data representing both directions of vehicle travel. The extrapolated results show very similar values with transducer S(1-2B), with the highest extrapolated event equal to approximately –125 µε.

The inverse normal plots for the two deflection transducers are illustrated in Figure 4.18. These graphs have again been plotted only for the data representing vehicle travel in the relevant direction. The maximum value for the 95% confidence limit in 100 years is 6.8 mm for transducer D(1-R). For the serviceability limit state (95% in 1 year) the deflection is 6 mm. The maximum results along with the extrapolated results for all transducers are presented in Table 4.1. The extrapolated data have been rounded. The extrapolated events (95% confidence limit for 100 years) range from 390 µε to 880 µε for the midspan transducers.

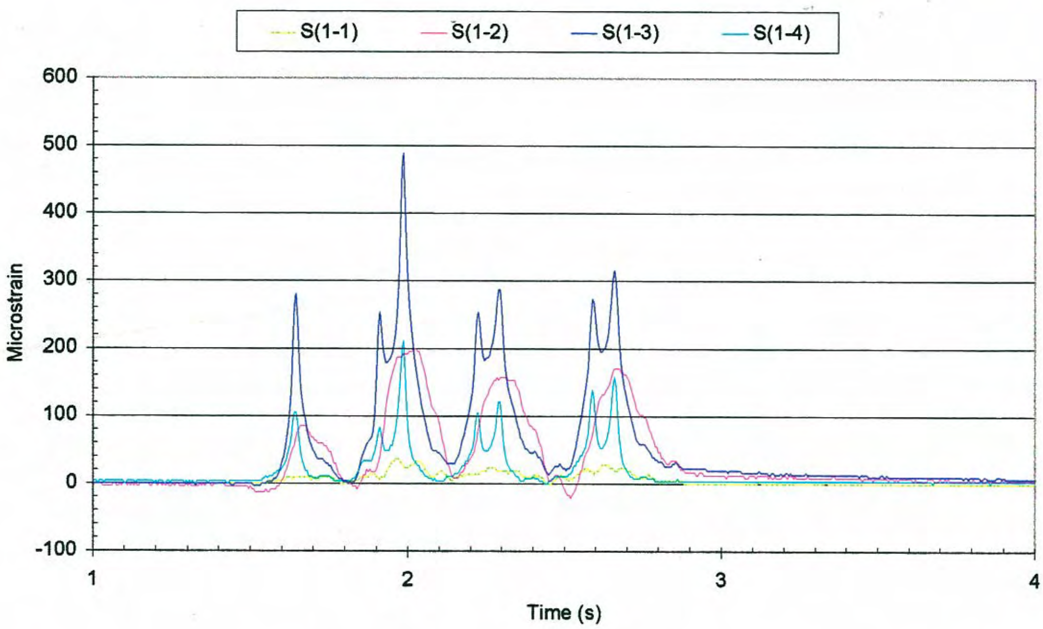
4.4 Known Vehicle Testing

The known vehicle testing was performed at different vehicle speeds ranging from a crawl to 70 km/h. A plot of the waveforms for each of the midspan strain transducers is presented in Figure 4.19 for the known vehicle travelling at 65 km/h towards Palmerston North.

Table 4.1 Extrapolated data obtained from inverse normal plot.

Transducer	Maximum Recorded Value (Health Monitoring)	Extrapolated Value (95% confidence limit) for 1 year	Extrapolated Value (95% confidence limit) for 100 years
	Strain ($\mu\varepsilon$)		
S(1-1)	230	405	390
S(1-2)	550	790	880
S(1-3)	510	775	870
S(1-4)	225	355	400
S(1-2A)	–65	–90	–100
S(1-2B)	–60	–100	–120
S(1-2C)	430	650	740
	Deflection (mm)		
D(1-L)	3.7	5.5	6.0
D(1-R)	4.0	6.0	7.0

Figure 4.19 Waveform for midspan strain transducers for the known vehicle travelling at 65 km/h towards Palmerston North.



The maximum strains that each transducer recorded during the testing with the known vehicle are presented in Table 4.2, and the distribution of load into each slab in terms of a percentage is presented in Figure 4.20. The distribution presented is relatively consistent with the data collected from Health Monitoring of the ambient heavy vehicle traffic. The results show very consistent results for the vehicle travelling in either direction.

Figure 4.20 Strain distribution for test with the known vehicle.

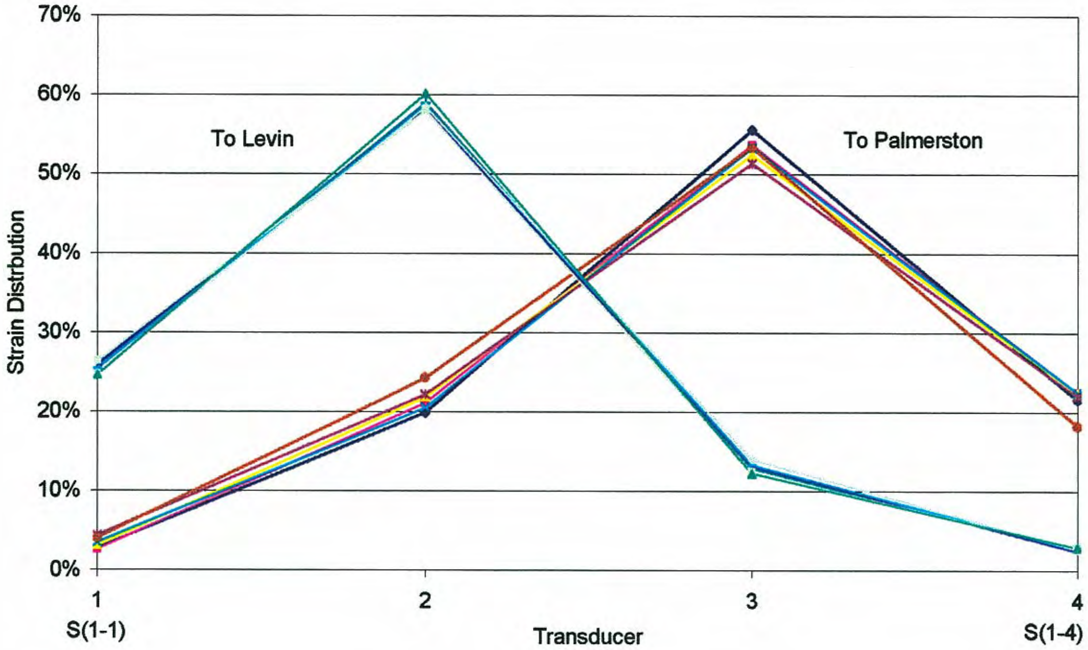


Figure 4.21 Absolute strain distribution for the known vehicle.

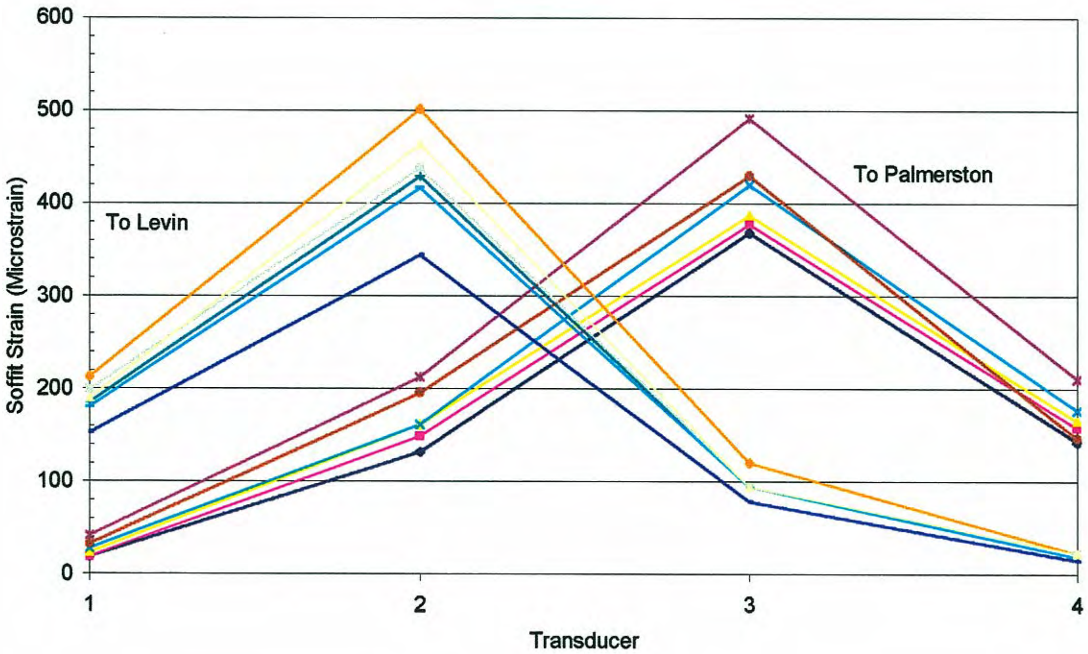


Table 4.2 Maximum responses (strains (με), deflections (mm)) recorded for known vehicle testing.

Transducer	Maximum Response
S(1-1)	213με
S(1-2)	502με
S(1-3)	492με
S(1-4)	211με
S(1-2A)	–44με
S(1-2B)	–39με
D(1-L)	3.2mm
D(1-R)	3.2mm

Figure 4.21 presents the distribution of strain into each slab in terms of the actual magnitudes. These results show a consistent response for the vehicle travelling in either direction.

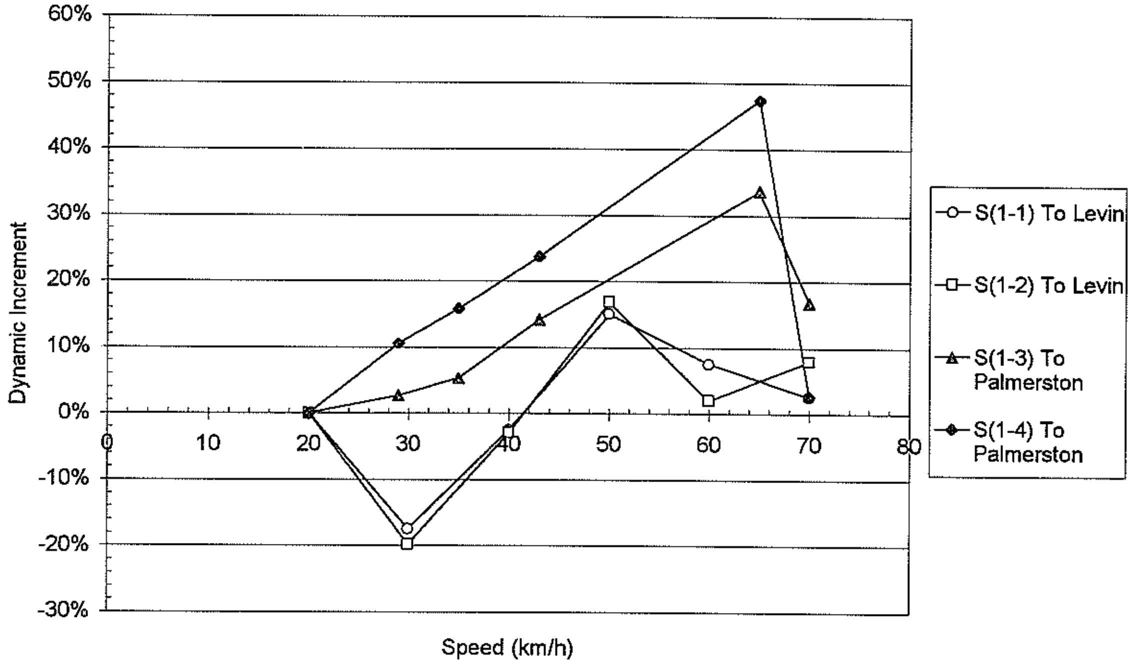
The dynamic response of this structure is very small as indicated by the waveform in Figure 4.19. Because this response is small, the level of damping and the natural frequency could not be determined. However the structure is highly damped and therefore resonant effects are not significant. Using higher resolution settings on the monitor would assist in identifying any dynamic response in the structure.

The dynamic increment (or impact factor) is used to indicate the increase in the effect of a vehicle on a structure as the speed increases. The dynamic increment (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 maximum dynamic increment for the Otauru Bridge was calculated for transducer S(1-4) and is equal to 47% (Figure 4.22). The difference between the dynamic increment for the vehicle travelling in opposite directions is evident, with the dynamic increment much higher for vehicles travelling towards Palmerston North. This difference is probably related to the poor road profile for that direction of travel. The impact factor recommended by the Bridge Manual for this bridge is 1.3. However the results in Figure 4.22 show that a much higher impact factor is appropriate for this bridge at service loads. The road profile at this site consists of a flat section of road over the bridge within an incline, and it is expected that changing the road profile so that it is a consistent incline would significantly reduce the dynamic effects.

Figure 4.22 Dynamic increment plot for the known vehicle test, for travel in the two directions, to Levin and to Palmerston North.



4.5 Summary

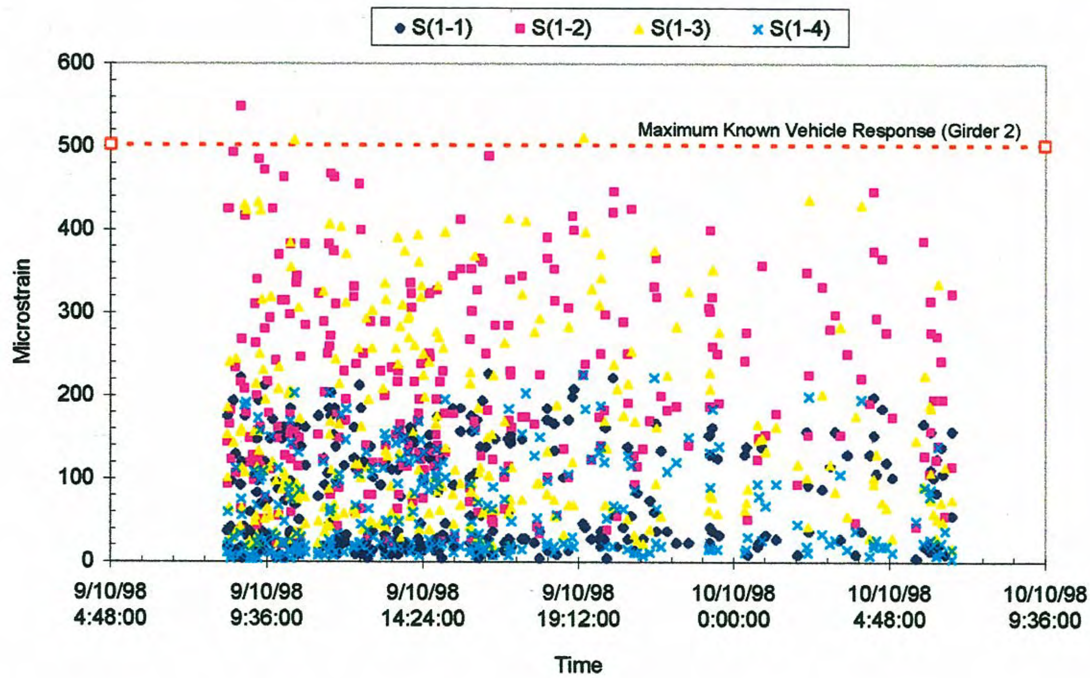
A summary of the data recorded for the Health Monitoring and the testing with the known vehicle is presented in Table 4.3. A comparison of the health monitoring data and the maximum response from the known vehicle is presented in Figure 4.23. This comparison shows that only one vehicle measured during the Health Monitoring programme caused a greater response than the known vehicle. This indicates that there is very little overloading on this route.

Generally the magnitude of the strains and deflections recorded are large for this type of structure. The dynamic testing with the known vehicle showed that overloading on this route is well controlled and that dynamic effects (impact factor) on this bridge are also large.

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 1 year)	Extrapolated Value (95% confidence limit for 100 years)
	Strain (με)			
S(1-1)	213	230	405	390
S(1-2)	502	550	790	880
S(1-3)	492	510	775	870
S(1-4)	211	225	355	400
S(1-2A)	–44	–65	–90	–100
S(1-2B)	–39	–60	–100	–120
S(1-2C)	–	430	650	740
	Deflection (mm)			
D(1-L)	3.2	3.7	5.5	6.0
D(1-R)	3.2	4.0	6.0	7.0

Figure 4.23 Comparison of response of maximum known vehicle with health monitoring data.



5. Fitness for Purpose Evaluation

5.1 Analysis of the Slab

The analysis described in section 3.2 of this report indicated that midspan bending of the slab was the critical mode of failure for the structure and the Fitness for Purpose has been evaluated based on this failure mode. The moment capacity available to resist the ultimate traffic live load at the midspan of the section is 110 kNm/m (ϕM_u – 1.25DL) and 272 kNm/m at the intersection of the slab and abutments.

5.1.1 Multiple Presence

The Otauru 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 this single span bridge 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 on the Otauru Bridge, based on the health monitoring data using the method that is consistent with the Bridge Manual. The diagram shows a transverse distribution of strain for each direction and the sum of these two distributions.

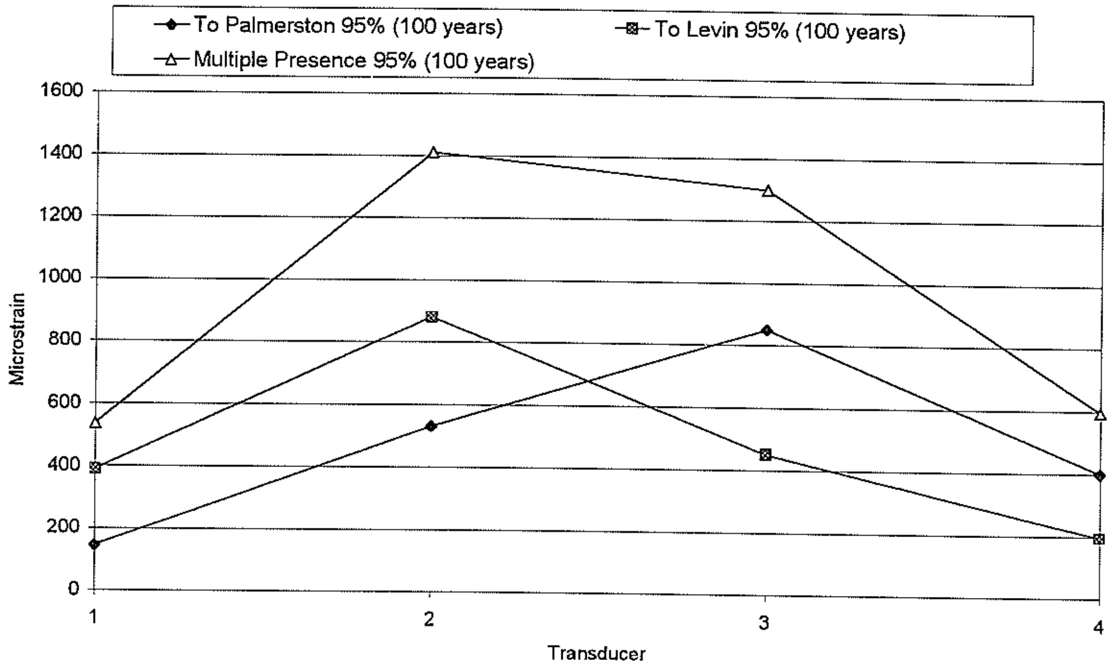


Figure 5.1 Multiple presence strains using the approach that is consistent with the Bridge Manual.

The maximum extrapolated health monitoring response for the midspan bending strain recorded on the soffit of the slab was 1400 $\mu\varepsilon$ for a multiple presence event. This strain was extrapolated from transducer S(1-2) which was a demountable strain gauge positioned over a crack in the slab. In this case, the measurement recorded by the demountable strain gauge represents the change in crack width related to the traffic live loads. The recorded data must therefore be adjusted based on crack width theory in order to obtain the actual tensile bending strain in the reinforcement in the slab.

5.1.2 Crack Width Theory

The crack width model used is based on the ACI[1] approach, as discussed in Warner et al. (1989). The maximum crack width (w_{max}) is based on the following relationship:

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

where:

σ stress in the reinforcement

h cover to bottom level of reinforcement

A concrete tension area surrounding reinforcing bars

Parameters are:

D overall depth of the section

d depth to centroid of reinforcement

K neutral axis parameter

1 ACI – Australian Concrete Institute

The maximum extrapolated event (95% in 100 years) is 1400 $\mu\varepsilon$ over a gauge length of 230 mm. This corresponds to a crack width movement (w_{max}) of 0.324 mm. Substituting this in Equation 8, along with the appropriate values of h and A for the concrete girder, gives a stress in the reinforcement of 350 MPa. This corresponds to a strain in the steel of 1750 $\mu\varepsilon$ and represents an increase in the strain in the steel compared with the strain recorded on the soffit of the slab. Consequently the recorded soffit strains in this report should be increased by 25% to represent the actual tensile bending strain in the reinforcing steel. Obviously the recorded strains are high and indicates that a plastic hinge has formed at this location.

5.1.3 Moment versus Strain Relationship

Figure 5.2 illustrates a theoretical moment versus strain curve for the slab of the Otauru Bridge. The graph summarises the method used by Infratech to obtain a relationship between bending moment and strain for determining the Fitness for Purpose Evaluation for this bridge. This is required because the relationship between bending moment and strain is not linear because the stiffness changes when the concrete slab cracks.

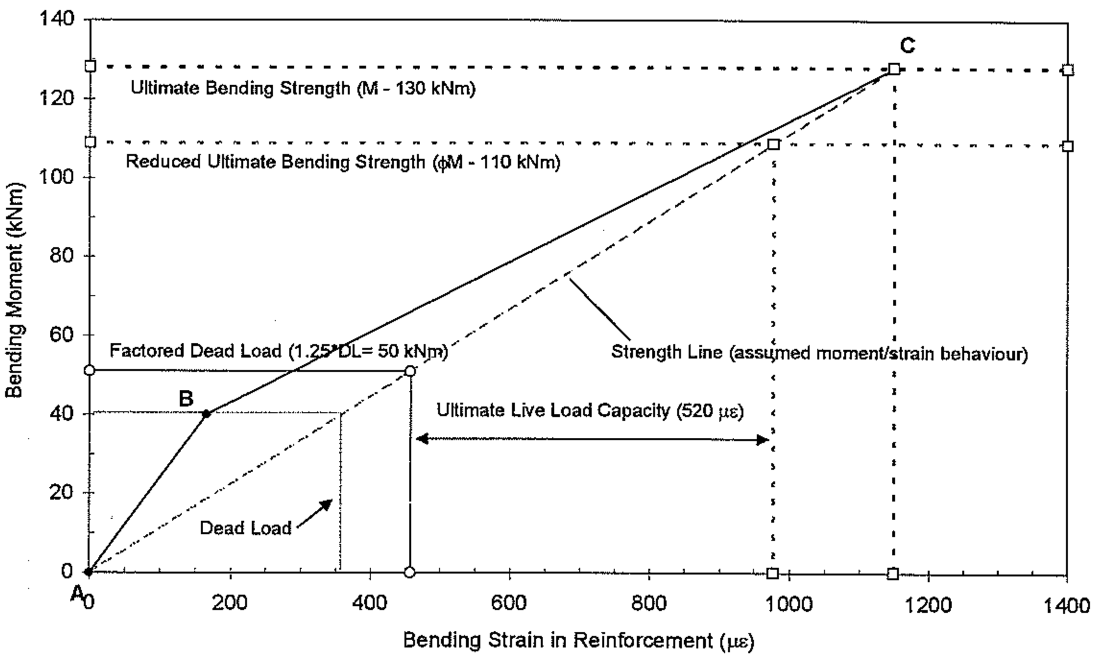


Figure 5.2 Theoretical moment versus strain relationship, and summary of Fitness for Purpose for the Otauru Bridge.

The line AB on Figure 5.2 represents the linear elastic behaviour of the concrete section before cracking occurs, and point B represents the point at which the concrete cracks. At this point the concrete begins to follow line BC which represents the behaviour of the concrete section in the cracked state.

A conservative assumption of the behaviour for the concrete section is characterised by line AC. Because the section is already cracked, under service loads the actual relationship between moment and strain for the girder is expected to be similar to the line AC.

Figure 5.2 also presents the reduced capacity (ϕM = 110 kNm) of a typical metre-width section of the slab for this bridge, converted to an equivalent strain in the reinforcement (987 $\mu\varepsilon$). This is based on the theoretical moment versus strain relationship in Figure 5.2. The factored dead load moment (50 kNm) was converted to an equivalent strain equal to 450 $\mu\varepsilon$, which gives an ultimate live load capacity equal to 520 $\mu\varepsilon$ when expressed in terms of strain.

5.1.4 Fitness for Purpose Evaluation

Table 5.1 summarises the calculation of the Fitness for Purpose Evaluation based on the health monitoring data and the information presented in Figure 5.2. The method for the calculation of this evaluation was outlined in section 2.4 of this report and involves dividing the ultimate live load capacity strain by the ultimate traffic load effect determined from the health monitoring data.

The Fitness for Purpose Evaluation for this bridge is 30%. This evaluation compares to the theoretical rating evaluation calculated for the 0.85 HO rating (75%) and for the 0.85 HN posting (95%) evaluations. The comparison with the 0.85 HN posting evaluation is the most appropriate as this loading is related to actual heavy vehicle traffic. The Fitness for Purpose Evaluation indicates that the structure is overstressed and intervention is required.

Table 5.1 Summary of Fitness for Purpose Evaluation.

Item	Result
Strength (ϕM)	110 kNm
Dead Load (*1.25)	50 kNm
Ultimate Live Load Capacity Moment ($\gamma_o R_o$)	60 kNm
Ultimate Live Load Capacity – Equivalent Strain ($\gamma_o R_o$)	520 $\mu\varepsilon$
Maximum Recorded Soffit Strain (Ambient Traffic)	550 $\mu\varepsilon$
Ultimate Traffic Load Effect (95% in 100 years) (Soffit Strain)	880 $\mu\varepsilon$
Ultimate Traffic Load Effect (95% in 100 years) (Multiple Presence - Soffit Strain)	1400 $\mu\varepsilon$
Ultimate Traffic Load Effect (95% in 100 years) (Reinforcement Strain)	1750 $\mu\varepsilon$
Fitness for Purpose Evaluation	30%

5.2 Summary

The strains recorded at midspan are high, and indicate that a plastic hinge may have formed at this location and that the structure is no longer performing as analysed. The structure is currently acting as a three-pinned arch, rather than as a portal frame on which the Fitness for Purpose Evaluation was based. The high strains recorded at midspan may lead to a fatigue failure of the plastic hinge, which may in turn lead to a gross failure of the structure.

The recommendation is to investigate this Otauru Bridge structure further and to take possible remedial action.

6. Conclusions

This report has presented the details and results of the Health Monitoring programme on the superstructure of the Otauru Bridge, which is essentially a large reinforced concrete culvert. A Fitness for Purpose Evaluation has also been derived for the bridge superstructure, based on the health monitoring data.

Theoretical Analysis

The theoretical analysis of the bridge found that midspan bending of the slab was the critical issue associated with the performance of the bridge, so the Health Monitoring programme focused on assessing the performance of the bridge based on this component. The ability of the concrete deck to resist wheel loads was not considered in this report.

The theoretical assessment of the superstructure of the bridge found that the 0.85 HO rating evaluation is 75% and the 0.85 HN posting evaluation is 95%. These compare with the value from the TNZ Structural Inventory of 95% for the 0.85 HO rating evaluation.

The comparison is relatively poor since Infratech's rating evaluation is 75% and the TNZ rating is 95%. However, this difference may be explained by variation in the assumptions used in the evaluations. The posting evaluation of 95% (i.e. <100%) means that the bridge should be posted with a load limit, although the need for this is marginal.

Health Monitoring Results

The findings of the Health Monitoring are that:

- High strains were recorded over the transverse crack at midspan indicating that a plastic hinge may have formed at midspan, and that the structure may not be acting as originally designed.
- The level of overloading on this route is well controlled.
- The dynamic increment or impact factor recorded for vehicles travelling towards Palmerston North is about 50%. This is much higher than the value recommended in the Bridge Manual of 30% for this structure. The high dynamic increment is caused by the poor road profile adjacent to the bridge.

Fitness for Purpose Evaluation

- The Fitness for Purpose Evaluation for this bridge based on the critical midspan bending of the slab was 30%, which indicates that intervention is required.
- The structure is apparently acting as a three-pinned arch, rather than as a portal frame on which the Fitness for Purpose Evaluation was based.
- The high strains at midspan may lead to a fatigue failure of the plastic hinge.

7. Recommendations

The recommendations are that:

- This structure requires remedial action.
- The Otauru Bridge should be further investigated
  - to determine if its current condition and performance is acceptable, and
  - to design and implement remedial action.
- Improvement of the road profile should be considered to reduce the effects of vehicles on this structure.

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.