Health Monitoring of Superstructures of New Zealand Road Bridges:

Waitangi Washout Bridge, Hawke's Bay

Transfund New Zealand Research Report No.171

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Waitangi Washout Bridge, Hawke's Bay

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Executive Summary

Introduction

Bridge Health Monitoring is a method of evaluating the ability of a bridge to perform its required task (also called Fitness for Purpose) by monitoring the response of the bridge to the traffic loads it has to withstand.

This report is part of Stage 2 of a research project carried out in 1998-1999, which involves the *Short-Term Health Monitoring and "Fitness for Purpose" Assessment* of ten bridges on New Zealand highways, in order to develop and evaluate the methodology.

The Waitangi Washout Bridge, on State Highway 2, crosses the Tutaekuri River between Napier and Hastings, Hawke's Bay Region, North Island. It was selected as one of these ten, and is an old (built in 1935), two-lane, three girder, reinforced concrete structure, with a low conventional strength evaluation.

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 analysis of the bridge found that midspan bending of the main girders and the performance of the deck were the critical issues associated with the performance of the bridge. The theoretical assessment of the superstructure of the bridge found that the rating evaluation was 65%, and the posting was 75%. The corresponding rating evaluation listed in the Transit New Zealand Structural Inventory was 61%. The Deck Capacity Factor (DCF) calculated for the deck was 1.22, compared with1.0 in the Structural Inventory.

Health Monitoring Results

The findings of the Health Monitoring programme were that:

- The heavy vehicle traffic on this route is inducing similar effects to that of the 0.85 HN* vehicle, with the exception of one vehicle which was significantly overloaded.
- The actual response of the bridge to heavy vehicles was significantly lower than the response predicted by the grillage model.
- The dynamic effects (probably related to the road profile) are influencing the effects of vehicles on this bridge. For example, the impact factor for vehicles travelling towards Hastings is significantly higher than that for vehicles travelling towards Napier.

* HN Highway normal vehicles; HO Highway overweight vehicles

Fitness for Purpose Evaluation

- The Fitness for Purpose Evaluation based on the main girders for this structure was 155%.
- The Fitness for Purpose of the deck was 230%, or 2.3 in terms of the DCF.
- The deck is therefore Fit for Purpose based on the heavy vehicle traffic that is currently using the structure. However in terms of the Bridge Manual this is not a true reflection of deck capacity, but it does reflect the actual loads being applied to the deck and the deck's ability to resist these loads.
- The Fitness for Purpose Evaluation of the superstucture of this bridge found that the main girders and the deck were Fit for Purpose.

Recommendations

The recommendations are that:

- A longer monitoring period is required to confirm the traffic characteristics on this route.
- The Bridge Manual does not allow the strength contribution from guardrails to be used in evaluations, and the relevant provision should be reviewed. Considerable benefit is gained by including the effect of guardrails, provided it can be adequately quantified.
- The differential behaviour between Girders 1 and 3 should be investigated. Although these two girders should nominally perform similarly to each other, Health Monitoring shows that their responses to similar loads vary considerably.

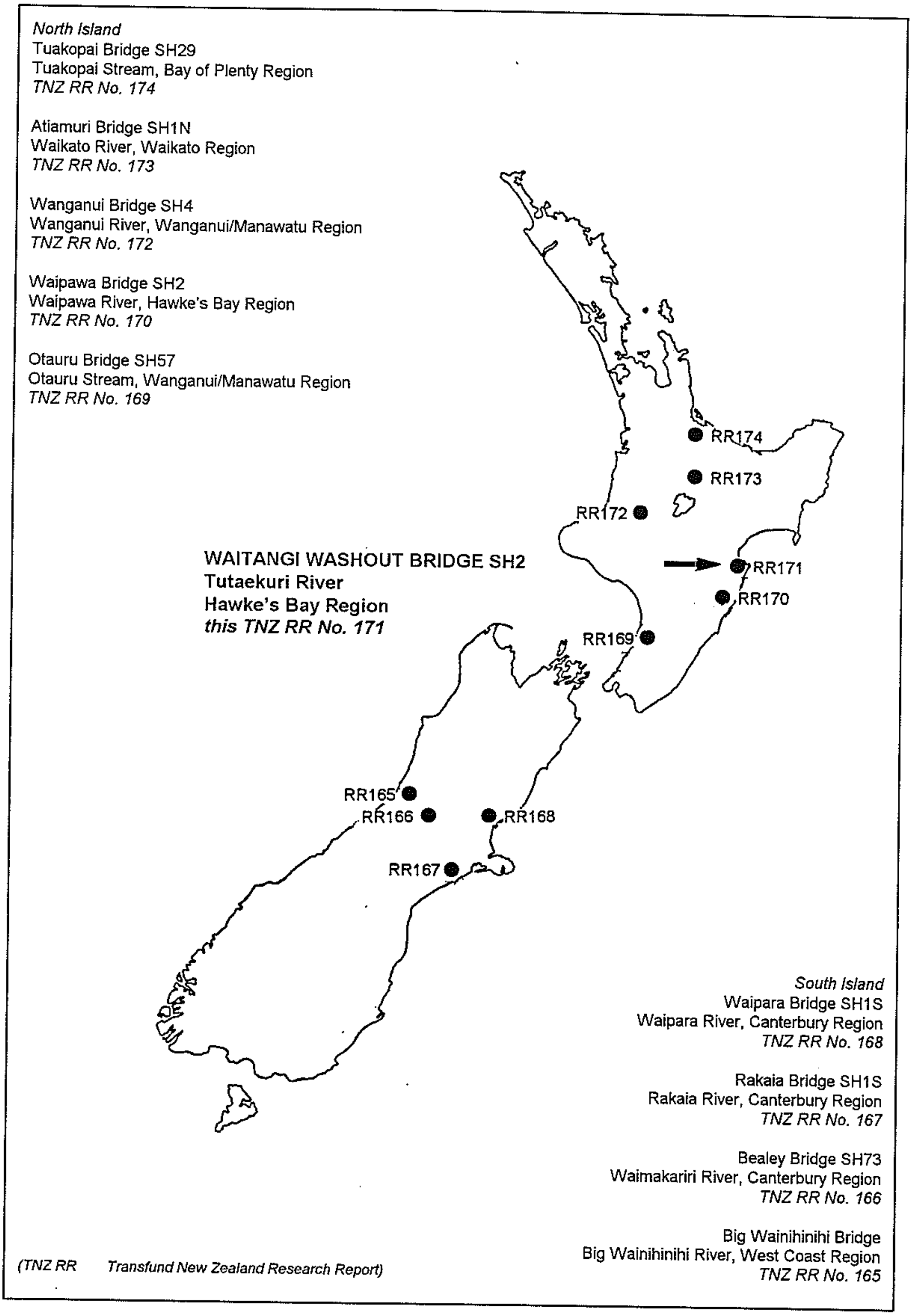
Abstract

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 Waitangi Washout Bridge, on State Highway 2, which crosses the Tutaekuri River, between Napier and Hastings, Hawke's Bay Region, North Island, was selected as one of these ten. It is an old (built in 1935), two-lane, three girder, reinforced concrete structure, with a low conventional strength rating.

Health Monitoring results show that the bridge performs substantially better, with more capacity than required, than the theoretical evaluations predict. Reasons for this improved performance are discussed in the report.

Figure 1.1 Location of Waitangi Washout Bridge, over Tutaekuri River, Hawke's Bay, North Island, New Zealand, one of the ten bridges selected for the Bridge Health Monitoring project.



1. Introduction

1.1 Background

Bridge Health Monitoring is a method of evaluating the ability of a bridge to perform its required task, also called its "Fitness for Purpose". This method involves monitoring the response of a bridge to its normal environment, in particular to the traffic loads it has to withstand. Subsequently 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, with conventional bridge theory and evaluation techniques. It has not been previously used in New Zealand as a systematic bridge evaluation technique, and consequently a project was conceived with the following objectives:

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

The project, conducted in 1998-1999, was divided into four stages, of which Stage 2 was entitled *Short-term Health Monitoring and "Fitness for Purpose" Assessment.* Short-term Health Monitoring was conducted on a total of ten New Zealand bridges on state highways, covering a range of bridge types, ages, conditions and environments. This population of ten bridges was selected to be representative of the New Zealand bridge population. It thus provided an appropriate basis to compare conventional bridge evaluation with the bridge Health Monitoring techniques under development. Not every aspect of every bridge has been considered, but rather the monitoring has typically focused on critical components of the superstructure of each bridge.

This report is part of Stage 2 of the project, and presents results for the Waitangi Washout Bridge, on State Highway (SH) 2, over the Tutaekuri River between Napier and Hastings, Hawke's Bay Region, North Island of New Zealand (Figure 1.1). The reasons for choosing this bridge for the representative sample were:

- It is old (built in 1935).
- It is a three girder, reinforced concrete structure.
- It has a low conventional strength rating.

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

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

The substructure was not evaluated in this investigation.

2. Evaluation of Bridges using Health Monitoring Techniques

2.1 Introduction

This section looks at the traditional approach to evaluating bridges as set out in the Bridge Manual (TNZ 1994). The advantages of a Health Monitoring approach are outlined, and a method to integrate the advantages of Health Monitoring in the existing evaluation procedures is also proposed.

Both bridge design and bridge evaluation involve ensuring that the probability of the load being greater than the resistance (i.e. the bridge fails) is acceptably small. This is illustrated graphically on Figure 2.1.

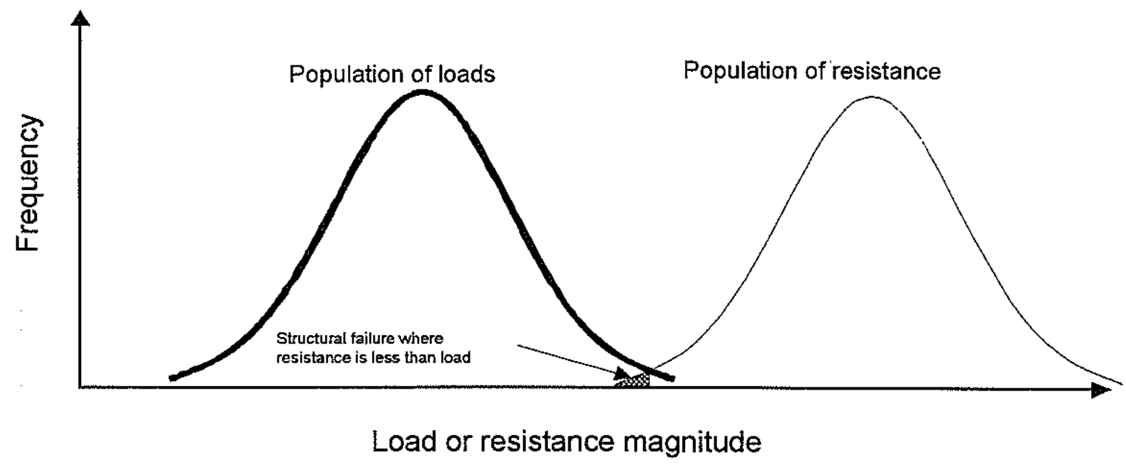


Figure 2.1 Statistical representation of structural failure.

Normally theoretical models are used to predict the magnitudes of loads and resistances in both design and evaluation processes. However, Health Monitoring utilises ambient traffic to investigate the effect that actual loads have on the in-situ structure. Thus the results of Health Monitoring provide an integrated measure of both the actual loads applied to the structure, and the effects that these loads have on the structure.

The objectives of bridge design and evaluation are similar, however the processes differ in some significant ways including:

- Bridge evaluation is more constrained than bridge design, since the infrastructure already exists in the latter case;
- Constraints are better understood during evaluation compared to design;
- Evaluation is usually associated with shorter time spans (typically 20 years compared to 100 years);
- Management options are often available and well understood during evaluations.

The estimation of structural resistance usually applies theoretical models based on engineering mechanics. Models of various levels of complexity are available, and these produce estimates of capacity with different levels of accuracy. Input data (material strengths, boundary conditions, etc.) are required for theoretical models, regardless of the model chosen. Much of these input data are based on a knowledge of construction procedures and tolerances. In the case of design, specific tolerances and parameters can be specifically controlled and confirmed where necessary.

When conducting evaluations however, greater uncertainty is usually associated with parameters (for example material strength). Conservative values can be chosen for the input data to allow for this, but will lead to under-estimation of capacity. Uncertainty may be reduced by testing all or part of the structure in some cases. Testing may also be important, because the resistance of an existing structure may decrease with time as physical deterioration progresses. In significantly deteriorated structures, this must be accounted for in the evaluation process.

Quantification of representative loads is generally more difficult than quantification of resistance, mainly because there is less control over bridge loading than there is over bridge construction and maintenance. In addition, design loads and legal loads are at best only indirectly linked. Design loads are generally developed by code writers who consider the worst-case loads likely to occur within the design life of structures. These loads are normally considered in two categories. The first is a set of loads intended to represent worst-case effects from normal legally loaded heavy vehicles (HN loading; TNZ 1994). The second is a set of loads intended to represent the worst-case effects from overloaded but permitted vehicles (HO loading; TNZ 1994). New bridges and their components are designed for the most severe effects resulting from both HN and HO loadings. This approach is intended to ensure that new bridges can accommodate current and foreseeable legal loads.

When evaluating existing bridges, there is limited scope to modify a bridge to change its capacity to accommodate future loads. However there is a strong need to understand its capacity to accommodate existing legal loads. The New Zealand Bridge Code (in TNZ 1994 Bridge Manual) empirically links legal loads with design loads for evaluation purposes. Essentially bridge evaluation loads are 85% of the design loads. If a bridge evaluation reveals that a given bridge cannot safely sustain 85% of the HO (overloaded/permitted legal heavy vehicle) loading, it will be **rated** consistent with its actual capacity to resist load. This rating will not be publicised, but will be used to approve or reject permit applications from transport operators requesting permission to cross the bridge with an overloaded (permitted) heavy vehicle. If a bridge evaluation reveals that a given bridge cannot safely sustain 85% of the HN (normal legal heavy vehicle) loading, it will be **posted** with a load limit consistent with its actual capacity to resist load.

2.2 Bridge Manual Evaluation Procedure

The Bridge Manual (1994) sets out the criteria for the design of new structures and evaluation of existing structures. Evaluation of existing structures is dealt with in Section 6 of that Manual. Existing bridges are typically evaluated at two load levels which are outlined below:

1. *A **Rating Evaluation** based on parameters to define the bridge capacity using overload factors and/or stress levels (i.e. appropriate for overweight vehicles).*

This evaluation is primarily concerned with evaluating the bridge's ability to carry overweight permit vehicles that comply with the Transit New Zealand Overweight Permit Manual (TNZ 1995), in a consistent and logical manner. However it is also used as a means of ranking and evaluating bridges for their capacity. This evaluation involves assessing the bridge's ability to carry a specific overweight vehicle load (0.85 HO loading).

2. *A **Posting Evaluation** based on parameters to define the bridge capacity using live load factors and or stress levels (i.e. appropriate for conforming vehicles).*

This evaluation is primarily concerned with evaluating the bridge's ability to carry vehicles which are characteristic of typical heavy vehicle traffic and comply with the TNZ Overweight Permit Manual (TNZ 1995). The evaluation involves assessing the bridge's ability to carry a design loading which is somewhat characteristic of typical heavy vehicle traffic (0.85 HN loading). If the bridge is unable to carry this loading, then the bridge is posted with the allowable load that the bridge can safely carry.

2.3 Member Capacity & Evaluation using TNZ Bridge Manual Criteria

The Bridge Manual deals with main members 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.

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

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 Waitangi Washout 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 Waitangi Washout 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 Waitangi Washout Bridge, located on State Highway (SH) 2, crosses the Tutaekuri River between Napier and Hastings, Hawke's Bay Region, North Island. The structure consists of twenty simply supported spans, each with three reinforced concrete girders supporting a reinforced concrete deck. The concrete deck has integral kerbs and concrete guardrails. The typical span length is 12.2 m. Construction of the 244 m-long bridge was completed in 1935. The bridge is illustrated in Figure 3.1, and Figure 3.2 shows the soffit of the superstructure of a typical span.

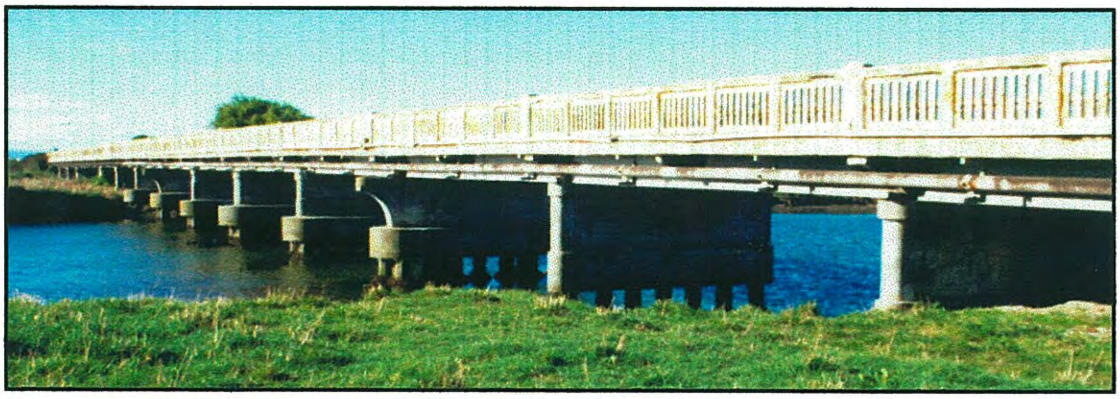


Figure 3.1 Waitangi Washout Bridge, in Hawke's Bay, North Island, New Zealand.

Figure 3.2 Typical superstructure soffit.

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

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

These ratings are based on the evaluation methods set out in Section 6 of the Bridge Manual, which are outlined in section 2.3 of this report.

3.2 Structural Assessment

To identify the critical failure modes of the superstructure, an analysis of the structure was conducted using the 0.85 HN and 0.85 HO posting and rating loads, as specified in the Bridge Manual (see section 2.1 of this report). 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 and that the guardrails do not contribute any bending strength to the structure. The dimensions of the structure used in the analysis were taken from the "as constructed" plans, and were confirmed by on-site measurements. The guardrails, which are integral with the deck, were not considered in the section properties for the grillage model. The kerbs however were included in the grillage model.

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

- Concrete f'_c = 17 MPa, E_c = 20 800 MPa
- Steel Reinforcement f_y = 250 MPa, E_s = 200 000 MPa

3.2.1 Girder Bending

The grillage analysis found that the edge girders were the critical components which determine the bending strength of this bridge. The maximum bending moment in the girders due to the dead load is 464 kNm/girder. A summary of the maximum bending moments resulting from the loads applied to the grillage model is presented in Table 3.1 for the critical edge girders. The results in the table are not factored and represent the extreme bending moment in a single girder with the vehicle at the greatest allowable eccentricity.

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

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

Load	Bending Moment (kNm)
Dead Load	464
Known Vehicle	264
2x 0.85HN Vehicles (Posting Load)	385
0.85HO + 0.85HN Vehicles (Rating Load)	567

The bending capacity (ϕM) of the concrete girders of the superstructure, calculated in accordance with Section 8 of the Concrete Structures Standard (NZS 3101: Part 1 1995), is 1320 kNm.

3.2.2 Girder Shear

The shear force in each girder from the grillage analysis is presented in Table 3.2. The shear capacity (ϕV) of the main girders, found in accordance with Section 9 of the Concrete Structures Standard (NZS 3101: Part 1 1995), is 655 kN.

Table 3.2 Results of grillage analysis for shear (kN) in centre girder.

Load	Shear Force (kN)
Dead Load	155
Known Vehicle	95
2x 0.85HN Vehicles (Posting Load)	136
0.85HO + 0.85HN Vehicles (Rating Load)	204

3.2.3 Deck Capacity

3.2.3.1 *Plate Bending*

The critical case for bending in the deck was determined using the Deck Evaluation Loads, given in Table 6.7 of the Bridge Manual. The loads include the twin-tyred load for the HN axle and both options of the HO axle loading (Section 3.1.2, Bridge Manual).

Analysis found that the HO wheel load was critical, with the resulting bending moment being 45 kNm, assuming the deck is continuous over the girders. The effective width of deck slab resisting this footprint was assumed to be 2 m, and the bending capacity of the deck at the ultimate limit state (ϕM) was 82 kNm.

3.2.3.2 *Shear*

The shear strength of the slab was calculated, using Section 9 of the Concrete Structures Standard (NZS 3101: Part 1 1995). The shear capacity of the deck was found to be 280 kN. The maximum shear force (V*) applied to the deck by the HO wheel loading is 80kN.

3.2.3.3 Empirical Method

The capacity of the deck can also be calculated using empirical methods, like that presented in the Bridge Manual (Section 6.5.2). This capacity is determined from Figures 6.1 to 6.5 in Section 6.5.2 of the Bridge Manual. For this bridge however, the empirical method is not applicable because the minimum strength of the concrete must be at least 20 MPa (and f'_c is assumed to be only 17 MPa).

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 superstructure are presented in Tables 3.3 and 3.4. The evaluation has been assessed for the bending and shear in the girders and deck. The table also presents a comparison of the evaluation calculated by Infratech Systems & Services (Infratech), and the evaluation recorded in the current (1999) TNZ Structural Inventory. The value of 1.3 was used for the impact factor, and a value of 1.3 was used for the dead load factor (γ_D) in determining the evaluations. The rating and posting loads presented in the table do not include impact factors. These are included in the rating and posting calculations.

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

Mode of Failure	ϕ Ultimate Capacity	0.85 HO Rating Load	0.85 HN Posting Load	Dead Load	0.85 HO Rating (Infratech)	0.85 HN Posting (Infratech)	Rating (Structural Inventory)
Girder Bending	1320kNm	567kNm	385kNm	464kNm	65%	75%	61%
Girder Shear	655kN	204kN	136kN	155kN	114%	135%	

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

Mode of Failure	ϕ Ultimate Capacity	HO Wheel Rating Load	HO Rating (DCF) (Infratech)	Rating (DCF) (Structural Inventory)
Deck Bending	82kNm (2m width)	45kNm	1.22	1.0
Deck Shear	280kN	80kN	2.35	

The overall evaluation of the girders is taken as the minimum evaluation of all the components. In this case the rating evaluation is the minimum of the evaluations based on shear and bending, which is 65%, and the critical failure mode is midspan bending of the girders. This compares to the rating evaluation of 61% which is documented in the TNZ Structural Inventory. The Deck Capacity Factor calculated by Infratech was 1.22, which is greater than the value of 1.0 found in the Structural Inventory.

The differences between the load evaluations calculated by Infratech and the TNZ Structural Inventory may be caused by differences in the assumed material properties and the assumptions relating to section properties in the grillage analysis.

Because the posting evaluation is less than 100%, the normal expected practice would be to post this bridge with a load limit. However, it is understood that this bridge is not currently posted.

3.4 Summary

The Waitangi Washout Bridge, in Hawke's Bay, was analysed using a grillage analysis to determine the bending moment and shear in the girders and deck of a typical span based on various vehicle loadings.

The bending moment in the girders was found to govern the strength and therefore determines the evaluation of the superstructure. The deck performance is governed by transverse bending at the midspan between the girders.

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

4. Health Monitoring Programme

The programme of Health Monitoring on the Waitangi Washout 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 Waitangi Washout Bridge.

4.1 Instrumentation

The instrumentation installed on the bridge included seven Demountable Strain Gauge transducers and one Deflection transducer. The instrumentation was installed on the fifth span of the structure from the southern end. The exact positions of this instrumentation are illustrated in Figure 4.1, and a cross section of the superstructure of the bridge is presented in Figure 4.2.

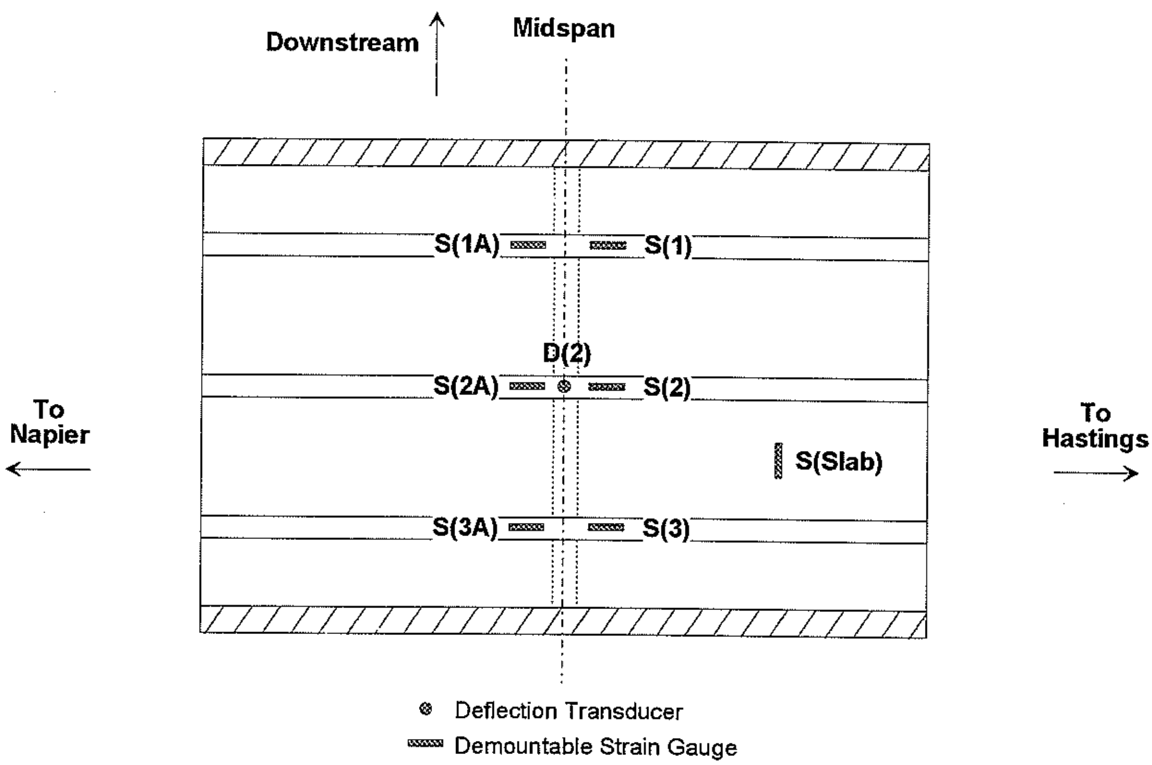


Figure 4.1 Instrumentation plan for Waitangi Washout Bridge (5th span from southern end).

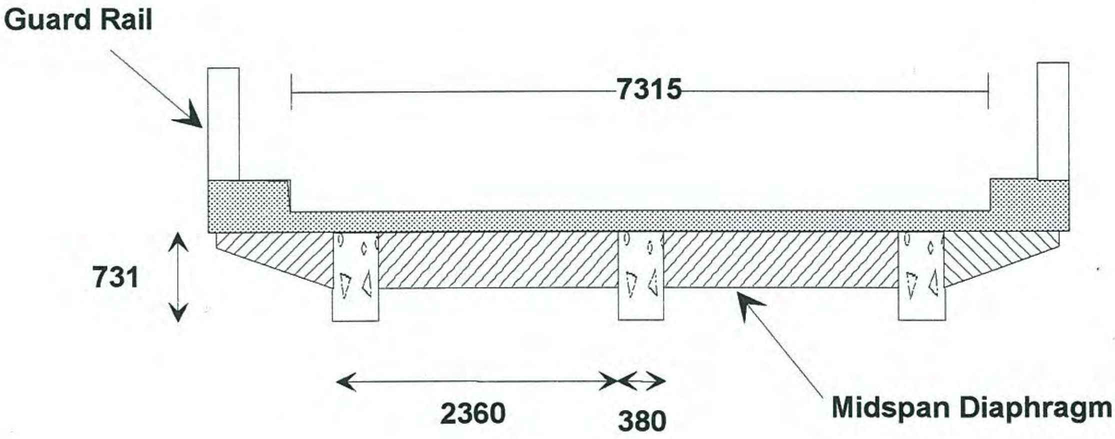


Figure 4.2 Cross section of Waitangi Bridge superstructure (dimensions in mm).

Figure 4.3 illustrates the positioning of the two demountable strain gauges and a deflection transducer installed on Girder 2. The rationale for using two demountable strain gauges at the midspan of each girder was to confirm the differences between strains measured over a crack, and strains measured in the concrete at the soffit where cracking was not evident. In each case the transducers labelled S(1A, 2A, 3A) were installed over a crack, and the transducers labelled S(1), S(2), S(3) were not installed over cracks. The deflection transducer was positioned to provide additional information on the performance of Girder 2.

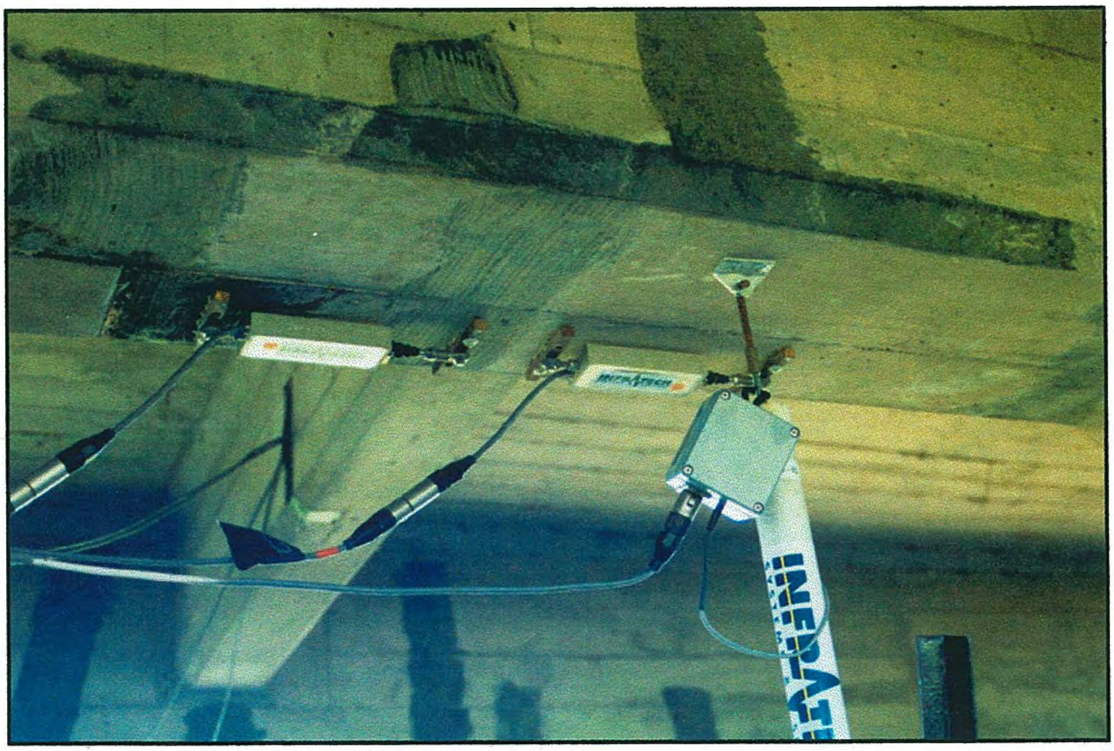


Figure 4.3 Instrumentation on Girder 2 of the bridge.

Transducer S(Slab) was installed to record the transverse bending strains in the deck. The transducer was positioned between Girders 2 and 3. Because the structure consists of three girders, the pathway of the wheel loads does not correspond well to the midspan of the deck in the transverse direction. Therefore the strains experienced in the deck may not be the maximum effect possible from the ambient heavy vehicles.

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

4.2 Procedure

The health monitoring of the structure began on Tuesday 6 October, and continued until Wednesday 7 October, 1998, giving a total monitoring period of approximately 24 hours. During the one-day monitoring period, the response of the bridge to 195 heavy vehicles was recorded, excluding the passage of the known vehicle.

In order to provide a control for all the data gathered during the entire monitoring period, the behaviour of the bridge in response to a known load (i.e. a heavy vehicle of known mass and dimensions) was measured. This component of the Health Monitoring programme was conducted on Wednesday 7 October, 1998. The known vehicle was a seven-axled heavy vehicle of known mass and dimensions, and was supplied by Renall Haulage Ltd, Masterton (Figure 4.4). The axle weights and configuration are illustrated in Figure 4.5, and the gross mass of the vehicle was 45.56 tonnes.

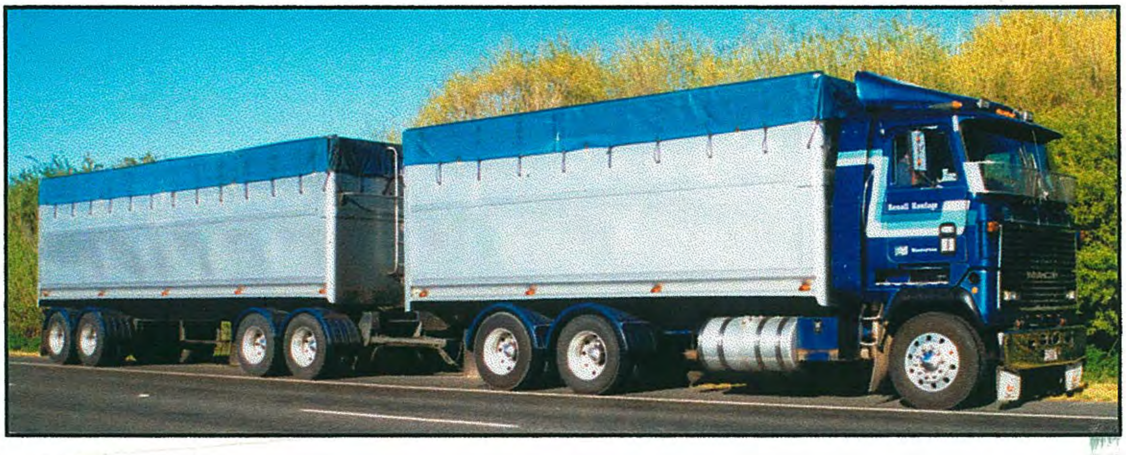


Figure 4.4 The known vehicle used for behavioural testing.

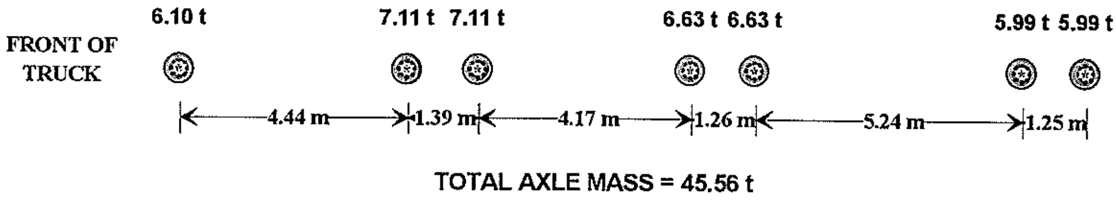


Figure 4.5 Axle mass and configuration of the known vehicle.

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, then north) from a crawl (20 km/h) to 80 km/h, in increments of 10 km/h.

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

4.3 Short-Term Health Monitoring Results

4.3.1 Girder Response

A typical strain response versus time was graphed (as waveforms in Figure 4.6) for the midspan bending strains recorded during the Health Monitoring, for the passage of a typical heavy vehicle. The waveforms show very little free vibration response after the vehicle has passed over the instrumented span. There is, however, some continuity between one adjacent span, as indicated by the compressive (negative) strains just before the vehicle passed over the instrumented span. This continuity is very small in comparison to the maximum strains and is not significant.

The waveform shows the large and similar response of the transducers installed on the two girders directly under the path of the vehicle. The transducers installed on the other side of the bridge show a small response indicating that very little load is being distributed across to the girder that is not loaded.

Figure 4.7 presents a scatter diagram for the maximum midspan bending strains recorded during the passage of each heavy vehicle for the entire Health Monitoring period, excluding the known vehicle testing. These plots give an indication of the characteristics of the heavy vehicles travelling over the bridge, as well as distribution of mass and the number of heavy vehicles travelling this route. The gap in the data on Wednesday 7 October 1998 is monitor down-time. The scatter plot shows that the vehicle represented by the waveforms in Figure 4.6 is significantly heavier than the other vehicles. The waveform characteristics for this event indicate that this event is of a single vehicle.

The difference in strains between positions that are so close together on the girder is primarily explained by the presence of cracks in the soffit of the girder. The strains recorded across the cracks are higher than the strains recorded in the adjacent area in the same girder where the transducer is not positioned across the crack. Therefore the positioning of transducers on concrete members is significant and it is recommended that, if the section is cracked, then the transducer should be installed over the cracks. The scatter diagram presented in Figure 4.7 indicates that, in some cases, the strains between adjacent transducers can vary by up to 10%.

Figure 4.8 Waveform for deflection transducer D(2) for event recorded at 6.20am, 7 October 1998 .

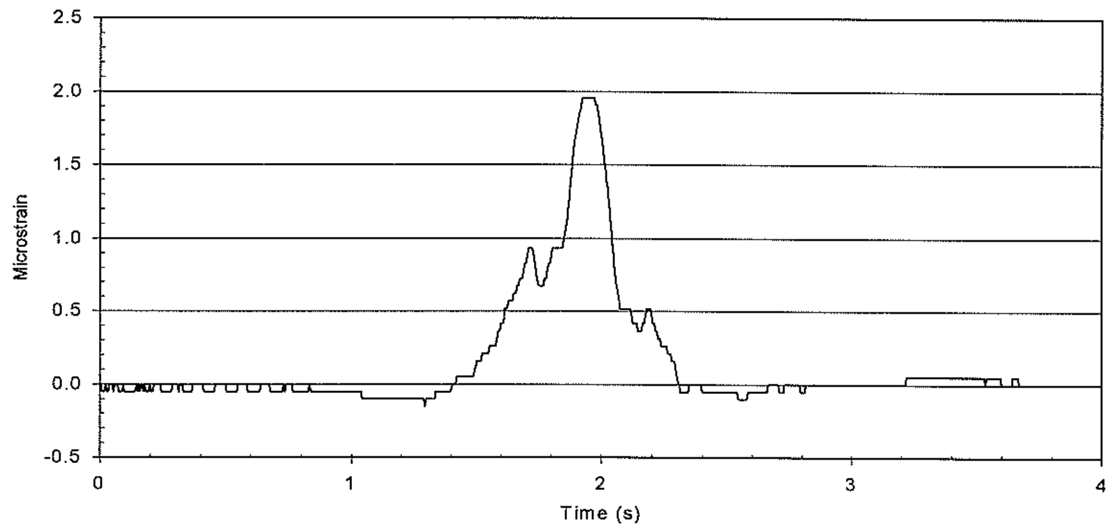
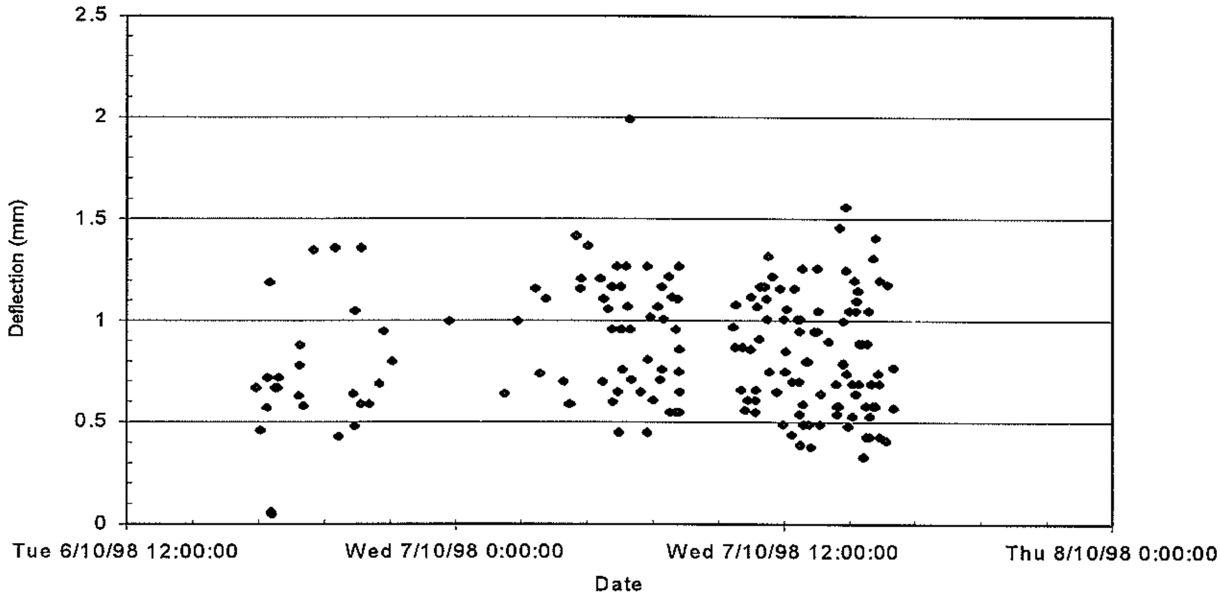


Figure 4.9 Scatter digram for deflection transducer D(2) for the entire monitoring period.



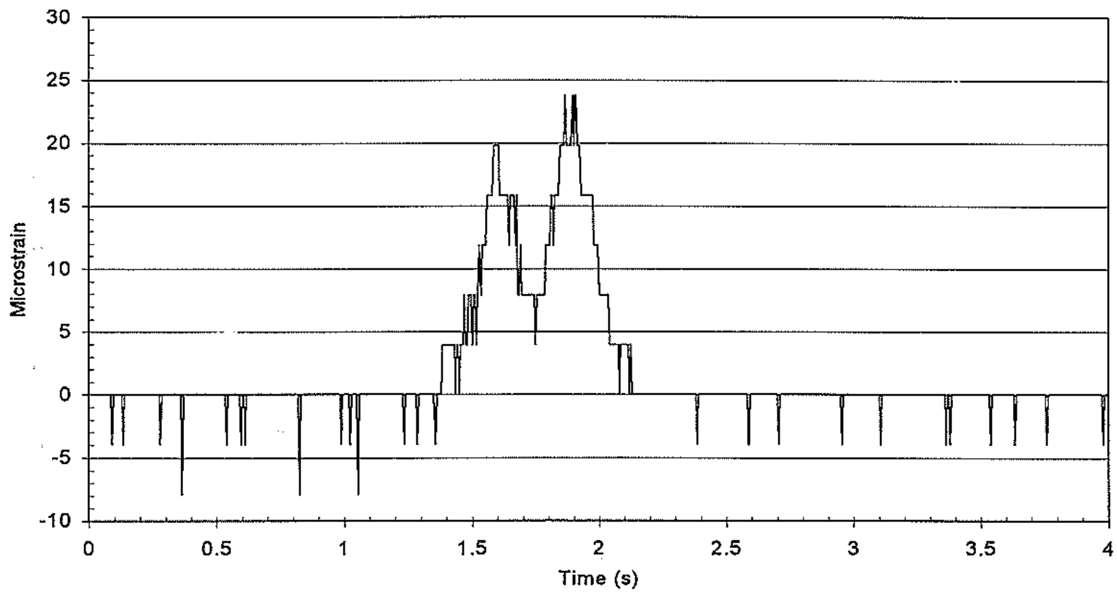
The scatter diagram also indicates that the strains experienced by Girder 1 are significantly higher than the strains in Girder 3. This is most probably explained by the relative overloading of vehicles in the direction of traffic flow corresponding to that side of the bridge. It is also possible that the variation is related to the relative condition of the girders. This issue will be discussed further in section 4.4 where the results for the testing with the known vehicle are presented.

A waveform for the deflection response of Girder 2 for the heaviest vehicle recorded is illustrated in Figure 4.8, and the scatter diagram for this transducer is presented in Figure 4.9.

4.3.2 Deck Response

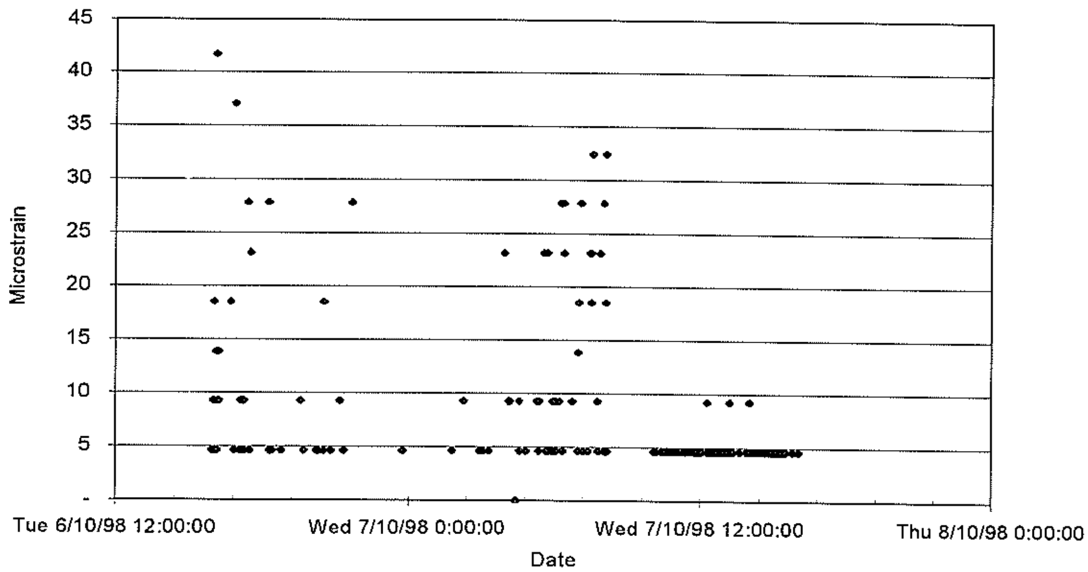
A typical waveform for the deck transducer is illustrated in Figure 4.10. The waveform shows the response of the deck as the wheel passes over the transducer. As the resolution of the waveform is very coarse, it does not produce a smooth waveform because the magnitude of the deck response is small. Higher resolution settings on the monitor would improve the quality of the recorded response.

Figure 4.10 Typical waveform for deck transducer S(Slab).



The scatter diagram for the deck transducer is presented in Figure 4.11. The deck and girder configuration of this bridge means that the normal traffic does not apply wheel loads to the critical location to induce the maximum strains in the deck. Increasing the monitoring period may increase the possibility of a wheel load being positioned at the critical location.

Figure 4.11 Scatter diagram for deck transducer S(Slab).



4.3.3 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 data recorded during the monitoring period can be separated into directions based on the lateral distribution of strain into the outside girders. By separating the data into directions, only the data relevant to each transducer can be plotted and a more accurate ultimate load effect can be determined.

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

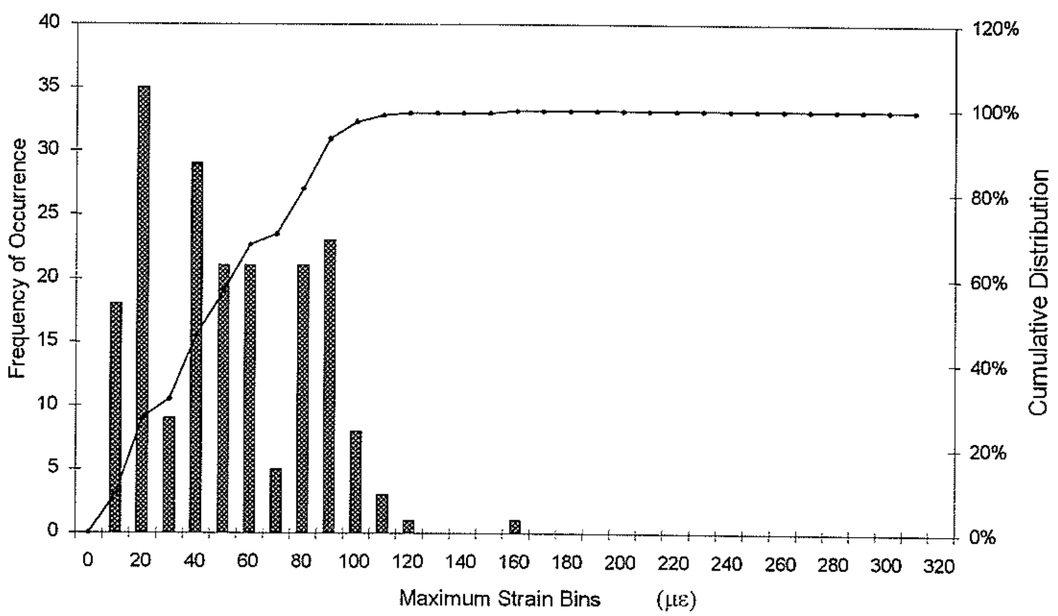


Figure 4.13 Inverse normal plot for strain transducers installed at the midspan.

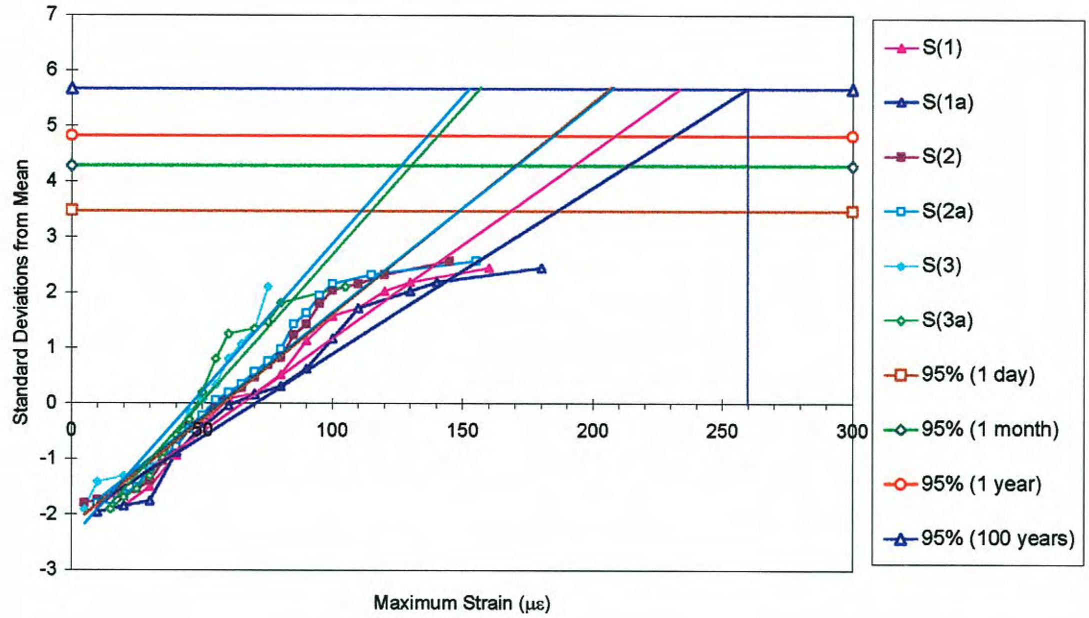
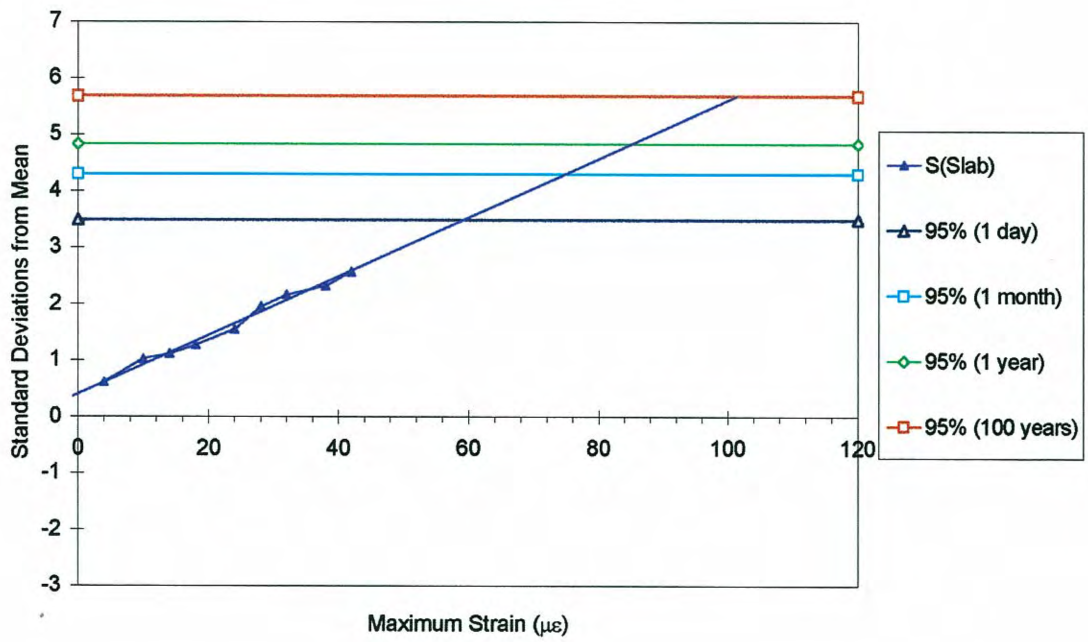


Figure 4.14 Inverse normal distribution for transducers installed on the deck.

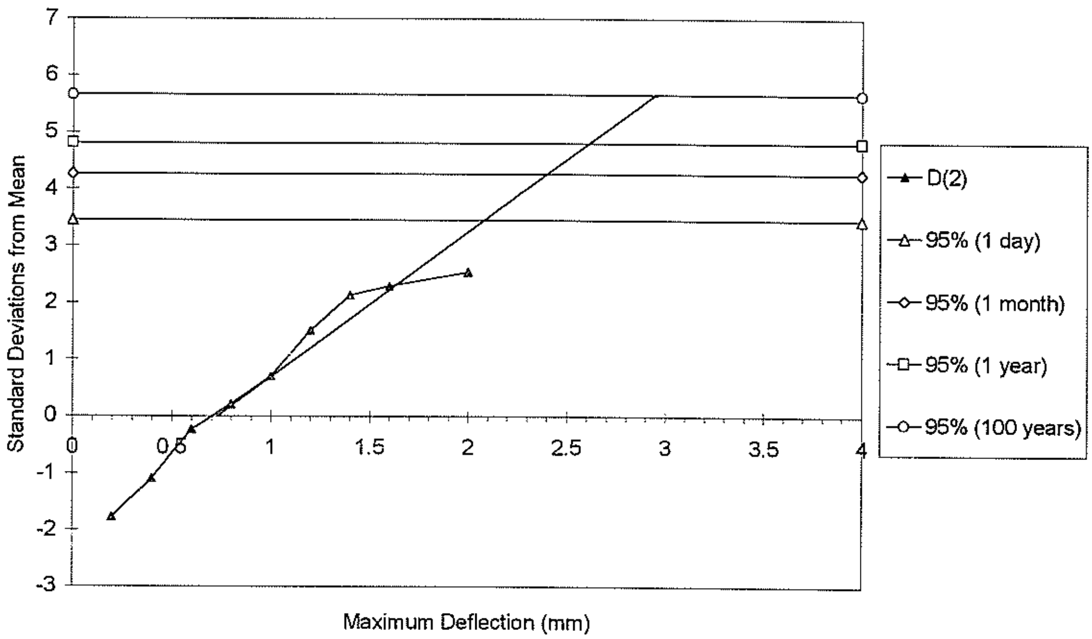


The cumulative distribution function can then be plotted on a probability scale known as an "inverse normal scale". The inverse normal plots for each of the transducers measuring midspan bending strain are presented in Figure 4.13. This figure presents the data that have been separated into the vehicle direction most affecting 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 the data plot crosses the horizontal axis represents the average (mean) strain, and 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.

This plot clearly shows the effect of the vehicle, which was significantly heavier than the other vehicles on the distribution. The plot also shows that the response from the traffic travelling towards Hastings (Girder 1 side) is larger than for the traffic travelling towards Napier (Girder 3 side). The inverse normal plot shows that the strain extrapolated for the 95% confidence limit for 100 years (ultimate traffic load effect) is the largest for the midspan transducer (S(1A)) on Girder 1. The extrapolated value is approximately 260 με.

Figure 4.15 Inverse normal plot for deflection transducer D(2).



The inverse normal plot for the transducer measuring transverse bending strain in the deck is presented in Figure 4.14, and the inverse normal plot for the deflection transducer installed on Girder 2 is presented in Figure 4.15. The results from the deflection transducer show similar behaviour to the strain results for Girder 2.

The maximum recorded results along with the extrapolated results for all transducers are presented in Table 4.1.

Table 4.1 Extrapolated data obtained from inverse normal distribution.

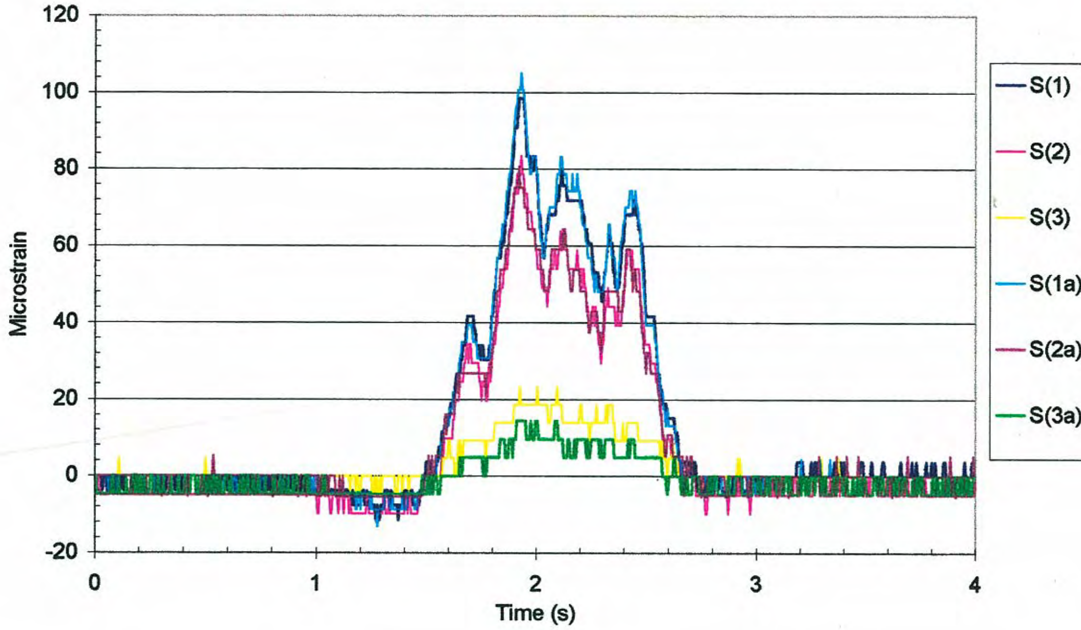
Transducer	Maximum Recorded Value (Health Monitoring)	Extrapolated Value (95% Confidence limit) for 100 years
	Strain ($\mu\varepsilon$)	
S(1)	160	225
S(2)	150	200
S(3)	75	140
S(1a)	175	260
S(2a)	155	200
S(3a)	105	145
S(Slab)	40	100
	Deflection (mm)	
D(2)	2.0	2.9

4.4 Known Vehicle Testing

4.4.1 Girder Response

A typical waveform from the testing with the known vehicle (travelling north to Hastings) is presented in Figure 4.16. The waveform suggests some degree of continuity between the spans. Structural actions are experienced in the instrumented span before the vehicle reaches the span. However the magnitude of this is not significant.

Figure 4.16 Typical waveform for passage of the known vehicle (travelling to Hastings).



The maximum strains for each transducer recorded from the known vehicle are presented in Table 4.2. These results indicate that the maximum responses of the transducers on Girder 1 are up to 50% larger than of the transducers installed on Girder 3. The structure is symmetrical and the load of the known vehicle on the structure was consistent in each direction of travel. Therefore the indication is that there is a difference in condition or structural behaviour between Girders 1 and 3, or that the condition of the road profile is increasing the load effect on the structure. This difference in Girders 1 and 3 is also confirmed by their significantly different strain distributions shown in Figure 4.17.

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

Transducer	Maximum Response
S(1)	98 με
S(2)	88 με
S(3)	65 με
S(1a)	105 με
S(2a)	81 με
S(3a)	53 με
S(Slab)	30 με
D(2)	2.5 mm

4.4.2 Strain Distribution

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 the ambient heavy vehicle traffic, The distribution shows that most of the strain was distributed into Girders 1 and 2 for the vehicles travelling to Hastings. This was almost identical for the vehicles travelling towards Napier, when most of the strain was distributed into Girders 2 and 3.

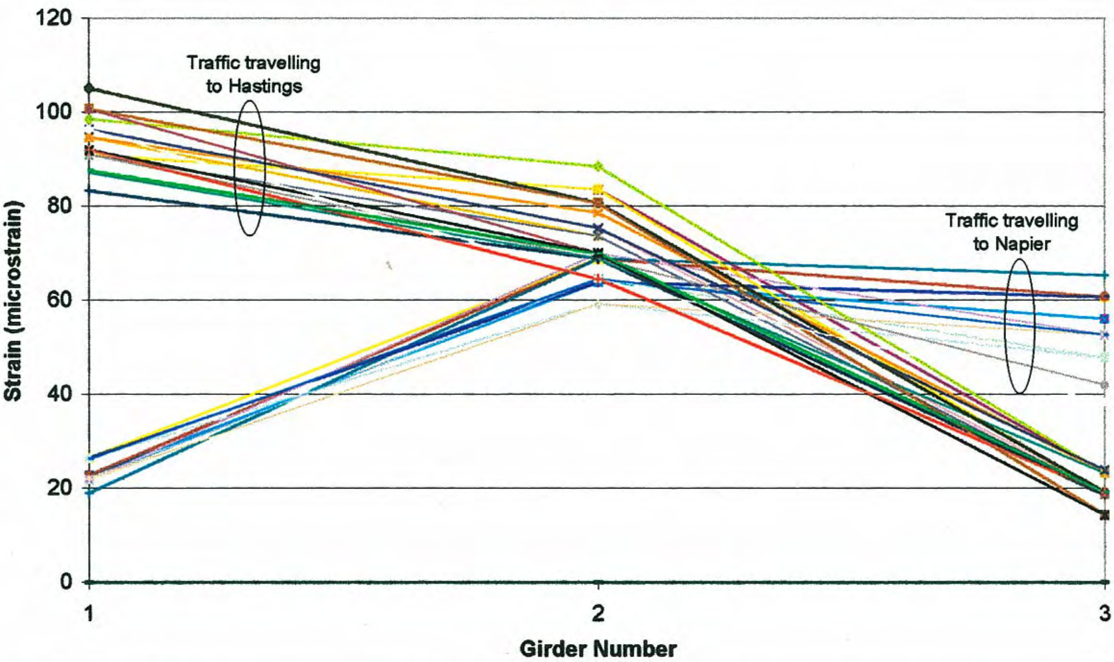


Figure 4.17 Distribution of strain for known vehicle testing for all girders, at different speeds.

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

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

The response of the crawl test (vehicle travelling at approximately 20 km/h) was used for the static result in the calculation of dynamic increment. The variation in dynamic increments for the known vehicle travelling to Hastings is illustrated in Figure 4.18 and for the vehicle travelling towards Napier in Figure 4.19. These graphs show a rise in the dynamic response as the vehicle increased speed towards Hastings. However the dynamic increment for the vehicle travelling towards Napier is less responsive, except at 40 km/h which appears to be related to a variation in lateral position of the vehicle, as the increase does not occur for both girders.

Figure 4.18 Dynamic increment plot for known vehicle travelling to Hastings.

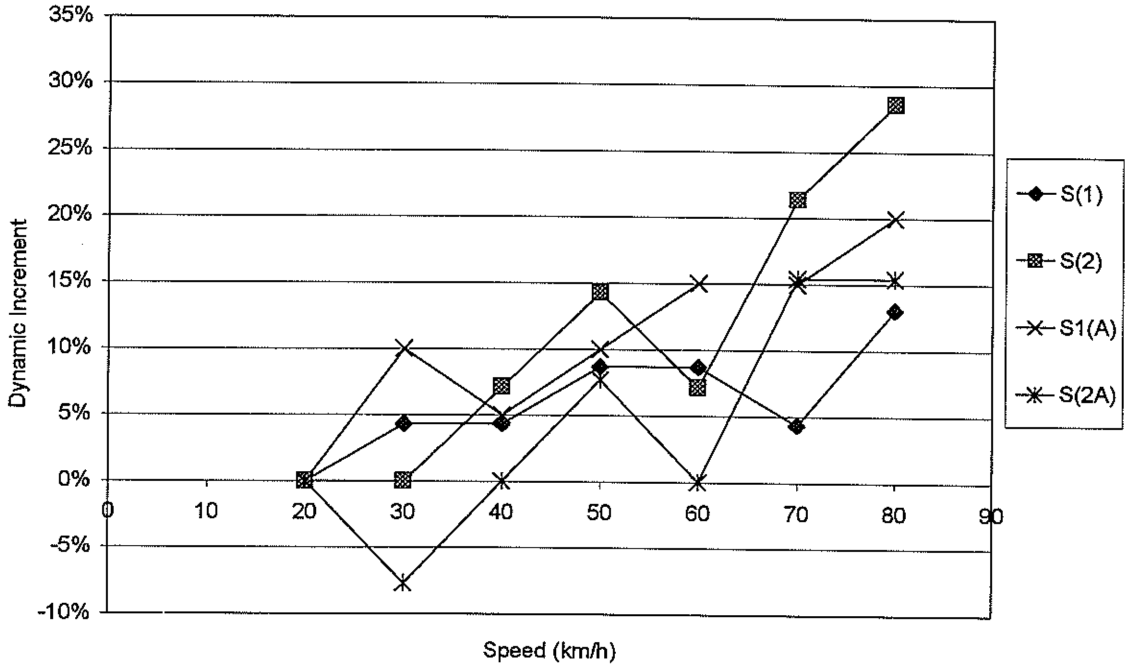
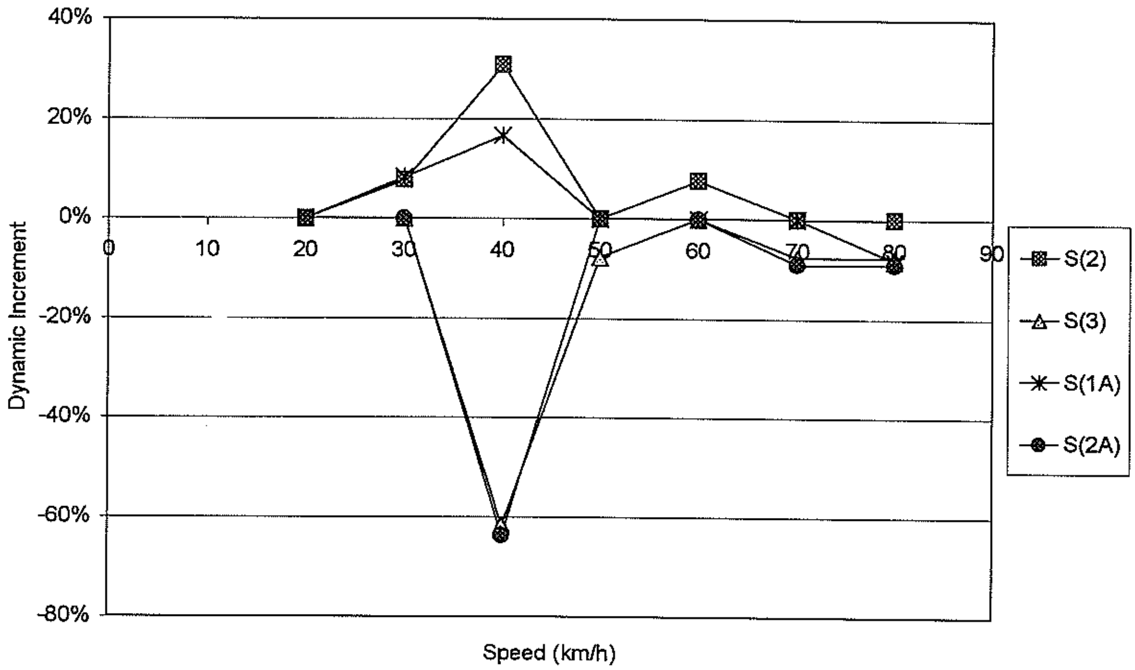


Figure 4.19 Dynamic increment plot for known vehicle travelling to Napier.



Only the transducers most effected by the passage of the vehicle are presented in these Figures 4.18 and 4.19. The maximum value of dynamic increment recorded was around 30%, which corresponds to the value recommended by the Bridge Manual for this span.

The free vibration response of the structure is not obvious from the waveform presented in Figure 4.16. Based on this response, the natural frequency and damping of the structure were not determined in this investigation. Higher resolution settings on the monitor would have allowed the free vibration response of the main girders to be determined.

4.5 Summary

Figure 4.20 illustrates the differences between the maximum recorded response of the structure to the known vehicle and the response of the ambient heavy vehicle traffic using the bridge. Data points above the dotted line represent events greater in magnitude than those related to the known vehicle. The greatest effects were experienced by Girder 1. This diagram shows approximately ten health monitoring events exceeding the known vehicle response.

A summary of the data recorded for the Health Monitoring and the testing with the known vehicle is presented in Table 4.3. The results for the maximum response of the structure to the ambient heavy traffic were up to 65% larger than the known vehicle. This difference is mainly due to the influence of only one vehicle in the traffic stream. In addition, some of the other vehicles were heavier than the known vehicle.

The Health Monitoring procedure and the testing with the known vehicle showed a significant difference between the response of Girders 1 and 3, and that this is mainly due to the effect of the road profile.

The recommendation is that a longer period of monitoring should be undertaken for this structure to determine if the heavy vehicle that was recorded as significantly heavier than the other traffic, was a one-off event or if it is typical of the traffic on this route.

5. Fitness for Purpose Evaluation

5.1 Main Girders

The structural assessment described in section 3.2 of this report indicated that midspan bending was the critical mode of failure for the Waitangi Washout Bridge. Thus the Fitness for Purpose Evaluation has been determined based on this failure mode. The moment capacity for the ultimate traffic live load is 720 kNm (ϕM – 1.3DL).

5.1.1 Multiple Presence

The Waitangi Washout 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 an instrumented span of the bridge during the monitoring period 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 methods are available. One is to simulate a multiple presence event by summing the 95% in 100 year event for a vehicle in each lane. 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 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 part of this project.

In this report the multiple presence effects have been determined using the approach that is consistent with the Bridge Manual. The distributions for the vehicles travelling to Hastings and to Napier are plotted in Figure 5.1 for the centre girder which is the critical component for multiple presence events. Both distributions have been extrapolated to the ultimate event corresponding to 95% in 100 years. The ultimate response for the vehicles travelling to Hastings is 210 $\mu\varepsilon$, and for the vehicles travelling to Napier the ultimate response is 170 $\mu\varepsilon$. Adding these events gives an ultimate multiple presence event with a magnitude of 380 $\mu\varepsilon$.

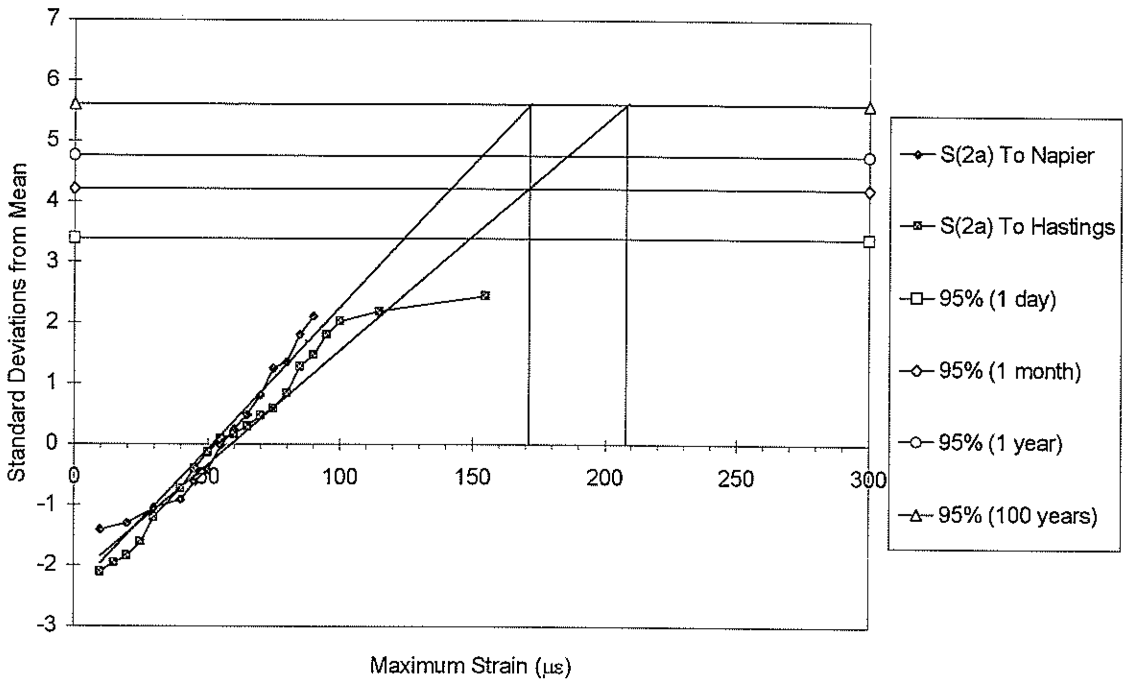


Figure 5.1 Inverse normal distributions used to determine the effect caused by multiple presence, using the approach consistent with the Bridge Manual.

5.1.2 Crack Width Theory

One of the objectives of Health Monitoring the Waitangi Washout Bridge was to present the variation in strains measured in concrete section which are cracked and uncracked. The girders in this bridge displayed a moderate degree of cracking at the midspan of the instrumented span. Strain gauges positioned over these cracks recorded higher strains than those positioned over uncracked sections.

The maximum response from the Health Monitoring procedure was obtained from transducer S(1A) which was positioned over a crack. In this case the measurement recorded by the demountable strain gauge represents the change in crack width caused by the traffic live loads. The recorded data must therefore be adjusted based on crack width theory in order to obtain the actual bending strain in the reinforcement in the girders.

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

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

Where:
σ_{st} stress in reinforcement
h cover to bottom level of reinforcement
A concrete tension area surrounding reinforcing bars

Parameters:
D depth of section
d depth to centroid of reinforcement
k neutral axis parameter

2 ACI Australian Concrete Institute

The maximum extrapolated multiple presence event (95% in 100 years) is 380 με over a gauge length of 230 mm. This corresponds to a crack width movement (w_{max}) of 0.089 mm. Substituting this into Equation 8 along with the appropriate values of h and A for the concrete girder gives a stress in the reinforcement of 73 MPa. This corresponds to a strain in the steel of 370 με and represents a decrease of approximately 3% over the recorded soffit strain. Consequently the recorded soffit strains in this investigation must be decreased by 3% in order to represent the actual bending strain in the reinforcing steel (i.e. the extrapolated multiple presence event in Girder 2 is 380 με, from Figure 5.1. However, this should be reduced to 370 με to allow for the effect of cracking).

5.1.3 Moment versus Strain Relationship

The maximum extrapolated multiple presence strain in the steel reinforcement (265 με) is well above the strain required to crack the concrete in the girders (160 με), and therefore the section must be considered to be behaving as a cracked section.

Figure 5.2 illustrates a theoretical moment versus strain curve for a typical girder of the Waitangi Washout Bridge. The graph presents the method used by Infratech to obtain a relationship between bending moment and strain for determining the Fitness for Purpose Evaluation for this bridge.

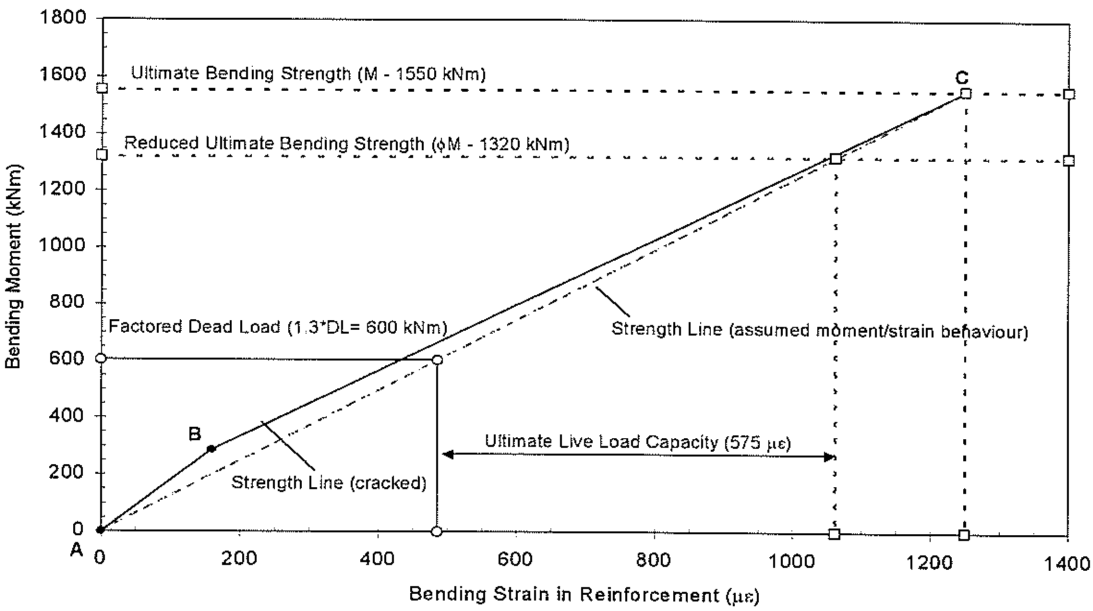


Figure 5.2 Moment versus strain relationship and summary of Fitness for Purpose for the Waitangi Washout Bridge.

Line AB on Figure 5.2 represents the linear elastic behaviour of the concrete girder. Point B represents the point at which the concrete cracks, at which point the moment strain relationship begins to follow Line BC. This represents the behaviour of the concrete girder in the cracked state.

Because the girders on this bridge are already cracked under service loads, the actual relationship between moment and strain for these girders is expected to be similar to line AC.

Figure 5.2 also presents the reduced capacity (ϕM) of a typical girder converted to an equivalent strain (1060 $\mu\varepsilon$), based on the theoretical moment versus strain relationship in the figure. The factored dead load moment (600 kNm) was converted to an equivalent strain equal to 485 $\mu\varepsilon$. This gives an ultimate live load capacity equal to 1060 – 485 = 575 $\mu\varepsilon$.

5.1.4 Fitness for Purpose Evaluation

Table 5.1 and Figure 5.3 summarise the calculation of the Fitness for Purpose Evaluation based on this data. The method for the calculation of this evaluation is outlined in section 2 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 155%. This evaluation is to be compared with the theoretical evaluation calculated for the rating load (65%) and for the posting load (75%). The comparison with the HN loading is the most appropriate because this loading is related to actual heavy vehicle traffic. The Fitness for Purpose Evaluation is much higher than the evaluation of the bridge based on the rating and posting loads.

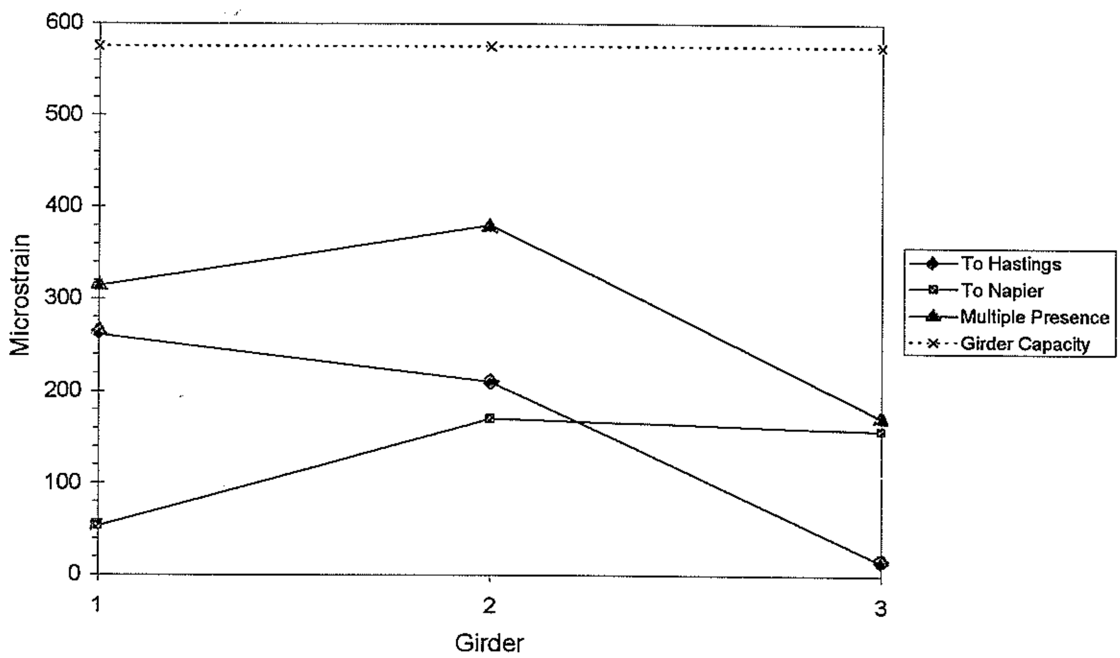


Figure 5.3 Summary of maximum strains for each Girder.

Table 5.1 Summary of Fitness for Purpose Evaluation.

Item	Result
Strength (ϕM)	1320 kNm
Dead Load (*1.3)	600 kNm
Ultimate Live Load Capacity Moment ($\gamma_o R_o$)	720 kNm
Ultimate Live Load Capacity – Equivalent Strain ($\gamma_o R_o$)	575 $\mu\varepsilon$
Maximum Recorded Soffit Strain (Ambient Traffic)	175 $\mu\varepsilon$ (Girder 1)
Ultimate Traffic Load Effect (95% in 100 years)	260 $\mu\varepsilon$ (Girder 1)
Ultimate Multiple Presence Load Effect (95% in 100 years)	380 $\mu\varepsilon$ (Girder 2)
Ultimate Multiple Presence Load Effect (strain in reinforcement)	370 $\mu\varepsilon$
Fitness for Purpose Evaluation	155%

5.2 Deck Slab

The evaluation of the deck slab capacity showed that the transverse bending capacity is determining the strength of the slab. The transverse bending capacity (ϕM) of the slab was 82 kNm. This corresponds to a bending strain in the soffit of the slab of 235 $\mu\varepsilon$, assuming that the slab behaves as a linearly elastic uncracked section. The ultimate traffic load effect for the concrete slab based on the health monitoring data is 100 $\mu\varepsilon$ and the resulting Fitness for Purpose Evaluation is 236%, or 2.36 in terms of a DCF. Again this is substantially higher than the value recorded in the TNZ Structural Inventory (1.0), and the value (1.22) calculated by Infratech. This evaluation indicates that the deck is Fit for Purpose.

The three-girder configuration of the structure (rather than the typical four girders), means that the normal wheel paths do not normally correspond to the midspan of the deck. Because of this, the Fitness for Purpose Evaluation of the deck is not a true reflection of the deck capacity as normally defined by the Bridge Manual. It does however reflect the actual loads being applied to the deck and the deck's ability to resist these loads.

5.3 Summary

The Fitness for Purpose Evaluation for the Waitangi Washout Bridge, based on midspan bending of the main girders, was 155%. The Fitness for Purpose Evaluation compares to the theoretical evaluation based on the 0.85 HN posting load of 75%. The following points summarise the findings of the Health Monitoring programme and the reasons for the differences between the evaluations:

- The known vehicle induces bending moments in the bridge that are equivalent to 85% of the 0.85 HN vehicle load.
- Most of the heavy vehicles induced effects that are similar or slightly higher to that of the known vehicle, with the exception of one vehicle which caused significantly greater effects. This vehicle may have been heavily overloaded.
- The actual response of the bridge to heavy vehicles was significantly lower than the response predicted by the grillage model. This lower than expected response is probably related to the effect of the concrete guardrails on this bridge. It is the principal reason for the high Fitness for Purpose Evaluation for this structure, compared with the theoretical evaluation.
- The dynamic effects (probably related to the road profile) are influencing the effects of vehicles on this bridge. The impact factor for vehicles travelling towards Hastings is significantly higher than the impact factor for vehicles travelling towards Napier.
- The Fitness for Purpose of the deck is 2.36. The deck is therefore Fit for Purpose, based on the heavy vehicle traffic that is currently using the structure.

As mentioned above, the main reason for the higher Fitness for Purpose Evaluation is the contribution of the concrete guardrails to the strength of the girders. This contribution should not be included according to the provisions of the Bridge Manual and therefore was not accounted for in the theoretical posting and rating evaluations in this report.

Health Monitoring, by its definition of measuring the actual response of the structure, includes the contribution of the guardrail. For this bridge the actual response of the structure was significantly less than predicted because of the contribution of the guardrails.

In the Fitness for Purpose Evaluation, the critical girder became the centre girder rather than the edge girder because of the contribution of the guardrails. To investigate their contribution, the guardrails have been instrumented on a similar bridge (Rakaia Bridge[3]) as part of this Stage 2 investigation.

[3] Transfund NZ Research Report No. 167 (Infratech 2000)

6. Conclusions

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

Theoretical Analysis

The theoretical assessment of the superstructure of the bridge found that the rating evaluation was 65% and the posting evaluation was 75%. These can be compared with the rating evaluation of 61% listed in the TNZ Structural Inventory. It is understood that, according to normal practice, this bridge would be posted based on this assessment.

Health Monitoring Results

The findings of the Health Monitoring were that:

- The heavy vehicle traffic on this route is inducing similar effects to that of the 0.85 HN vehicle, with the exception of one vehicle, which may have been significantly overloaded.
- The actual response of the bridge to heavy vehicles was significantly lower than the response predicted by the grillage model.
- The dynamic effects (probably related to the road profile) are influencing the effects of vehicles on this bridge. The impact factor for vehicles travelling towards Hastings is significantly higher than the impact factor for vehicles travelling towards Napier.

Fitness for Purpose Evaluation

- The Fitness for Purpose Evaluation based on the main girders for this structure was 155%. This Health Monitoring evaluation is significantly greater than the theoretical evaluations noted above.
- The Fitness for Purpose of the deck was 230%, or 2.3 in terms of a DCF. The deck is therefore Fit for Purpose based on the heavy vehicle traffic that is currently using the structure.

  These higher evaluations are expected to be mainly related to the effect of the concrete guardrails on this bridge,

  However this result is not a true reflection of the deck capacity for this bridge in terms of the Bridge Manual, because the normal wheel paths do not normally correspond to the midspan of the deck. It does however reflect the actual loads being applied to the deck and the deck's ability to resist these loads.
- The Fitness for Purpose Evaluation of the superstructure of this bridge found that the main girders and the deck were Fit for Purpose.

7. Recommendations

- A longer period of monitoring is recommended to determine if this vehicle was an isolated event or whether it is more characteristic of the traffic on this route.
- The Bridge Manual does not allow the strength contribution from guardrails to be used in evaluations, and the relevant provision should be reviewed. Considerable benefit is gained by including the effect of guardrails, provided it can be adequately quantified.
- The differential behaviour between Girders 1 and 3 should be investigated. Although these two girders should nominally perform similarly to each other, Health Monitoring shows that their responses to similar loads vary considerably.

8. References

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