Health Monitoring of Superstructures of New Zealand Road Bridges:

Big Wainihinihi Bridge, West Coast

Transfund New Zealand Research Report No. 165

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Big Wainihinihi Bridge, West Coast

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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.

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Executive Summary

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

This report is part of Stage 2 of a research project carried out in 1998-1999 which involves the *Short-Term Health Monitoring and Fitness for Purpose Assessment* of 10 bridges on New Zealand highways, to develop and evaluate the methodology. The Big Wainihinihi River road bridge, on State Highway 73, between Kumara and Otira, West Coast Region, South Island, New Zealand, was built in 1935, and was selected as one of these ten because it is an aging single-lane steel-girder bridge with a timber deck. The conventional rating for it is relatively low, and it is also representative of a large number of road bridges maintained by local government agencies throughout New Zealand.

This report details a theoretical assessment of the bridge to determine the critical elements for the Health Monitoring, and the Fitness for Purpose evaluation for the bridge, based on the health monitoring data. This assessment and evaluation considers only bending and shear strength in the main girders and the bending strength of the deck.

The theoretical analysis of the bridge found that midspan bending of the main girders and the performance of the deck were the governing parameters associated with the performance of the bridge. The interpretation of the restraint of the girders at the diaphragms is a critical issue. The girder capacity calculated in this report is based on the assumption that the girders are fully restrained at the diaphragms, and the recommendation is to review the restraint conditions in line with this assumption.

The theoretical assessment of the superstructure of the bridge found that the 0.85 HO[1] rating evaluation was 59%, the 0.85 HN[1] posting evaluation was 84%, and the Deck Capacity Factor (DCF) was 1.3 for the deck.

The findings from the Health Monitoring include:

- The girders act as simply supported members and the timber deck does not act compositely with the girders.
- The ambient heavy vehicle traffic is typically inducing lower bending moments than the 0.85 HN vehicle for the monitored span. The known vehicle (i.e. of known mass and dimensions, used for testing) induced bending effects that are similar to that of the 0.85 HN vehicle loading, and a significant proportion of the ambient heavy vehicle traffic induced effects that are less than this vehicle. Consequently the level of overloading on this bridge is well controlled and the number of heavy vehicles is low.
- Because the vehicles generally drive down the centre of this bridge (as it has only one-lane), the distribution of load into the edge girders is not as excessive as the theoretical analysis predicts.

[1] HO Highway overweight vehicles; HN Highway normal vehicles

The Fitness for Purpose evaluation for this bridge, based on critical midspan bending of the main girders, was 107 %. The Fitness for Purpose Deck Capacity Factor was 2.0. However this figure was based on a number of assumptions that would require verification. This evaluation indicates that the bridge is safely carrying the heavy vehicle traffic currently using the route. The Fitness for Purpose evaluation is higher than the 0.85 HN posting evaluation, principally because of the factors outlined above.

The Fitness for Purpose evaluation was based on the assumption that the main steel girders are laterally braced at the third point bracing locations. The true nature of the restraint provided to the girders by the decking system is critical to girder capacity, but is unclear for this bridge. Consequently it is recommended that the restraint of the girders should be reviewed. In this context the following should be noted:

- If adequate (third point) restraint is not present, the capacity of the bridge would be significantly reduced below that used in the Fitness for Purpose evaluation;
- If restraint is significantly better than that assumed (third point), then the bridge capacity could be substantially increased above that used in the Fitness for Purpose evaluation.

The two recommended approaches to resolve the issue of lateral restraint of the girders are:

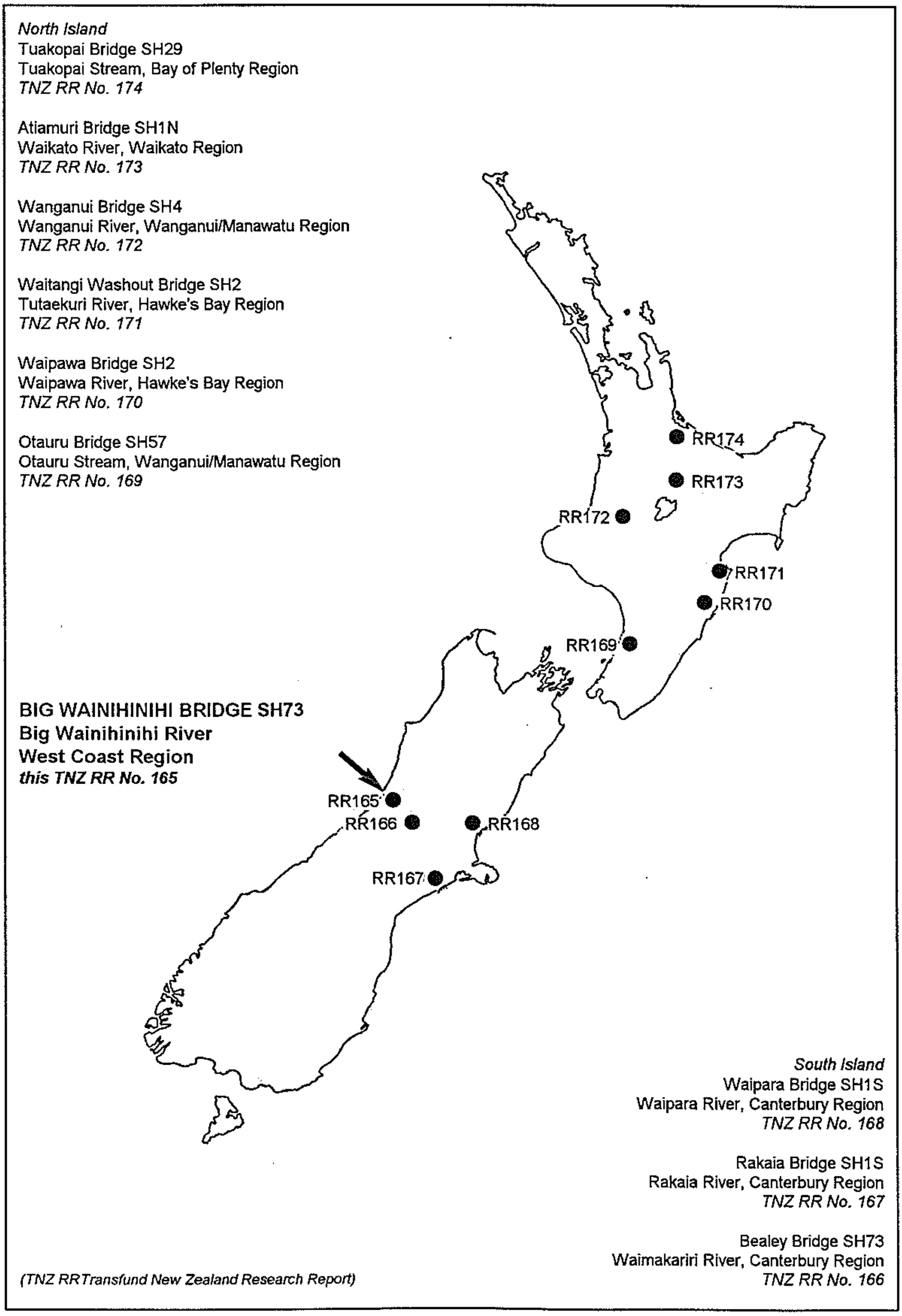
- Test the bridge to determine the lateral restraint characteristics: this would involve some additional health monitoring, and the use of a known heavy vehicle;
- Stiffen the third point restraints of the girders to ensure that they provide effective restraint: this could involve adding horizontal bracing members to the girder bottom flanges at diaphragm locations.

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 the *Short-Term Health Monitoring and "Fitness for Purpose" Assessment* of 10 bridges on New Zealand roads, to develop and evaluate the methodology. The Big Wainihinihi River road bridge, on State Highway 73, between Kumara and Otira, West Coast Region, South Island, and built in 1935, was selected as one of these ten because it is an aging single-lane steel-girder bridge with a timber deck. The conventional rating for it is relatively low, and it is also representative of a large number of road bridges maintained by local government agencies throughout New Zealand. However, the Fitness for Purpose evaluation for this bridge indicates that the bridge is safely carrying the heavy vehicle traffic currently using the route.

Figure 1.1 Location of Big Wainihinihi River bridge, South 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. This method involves monitoring the response of a bridge to its normal environment, in particular its traffic loading. Subsequently these data are processed, and used to evaluate the bridge's "Fitness for Purpose".

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

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

The project was divided into four stages. 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.

This report is part of Stage 2 of the project, and presents results for the Big Wainihinihi River bridge, on State Highway 73 (SH73), between Kumara and Otira, West Coast Region, South Island, New Zealand (Figure 1.1). The reasons for choosing this bridge in the representative sample were:

- It is aging (built in 1935);
- It is typical of single lane steel-girder bridges with timber decks, in New Zealand;
- Its conventional capacity rating was low indicating that its Fitness for Purpose was questionable;

- It is representative of many similar bridges maintained by local government agencies, as well as by state authorities, throughout New Zealand.

The objective of this report was to evaluate the Fitness for Purpose of the Big Wainihinihi River bridge superstructure, using the conventional evaluation technique and the proposed health monitoring technique, then to compare the results of both techniques. Specifically, the fitness of the bridge to carry heavy vehicle traffic loadings was investigated.

1.2 Applying Health Monitoring Technology

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

- Performing a structural analysis on the bridge superstructure: this identifies the most appropriate instrument locations for health monitoring;
- Monitoring the response of the superstructure 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.

Subsequently the Health Monitoring evaluation was compared with the conventional procedure.

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

- Midspan bending strength of the main steel girders;
- Shear strength of the main steel girders;
- Bending strength of the timber deck.

The bridge sub-structure 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.

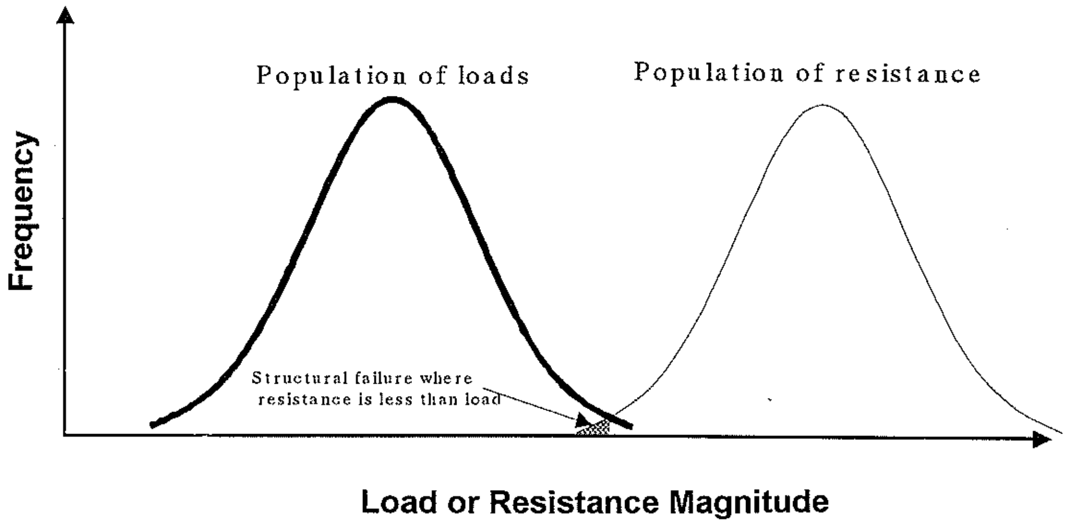


Figure 2.1 Statistical representation of structural failure.

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 uses 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.) is required for theoretical models, regardless of the model chosen. Much of this input data is 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 considering 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) 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 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 the two load levels outlined below:

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

This evaluation is concerned with evaluating the bridge's ability to carry overweight permit vehicles as per the TNZ 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** using parameters to define the bridge capacity using live load factors and/or stress levels (i.e. those 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). These vehicles comply with normal legally loaded vehicles. 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 Section Capacity and Rating using TNZ Bridge Manual Criteria

The Bridge Manual deals with main members and decks of the bridge separately.

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. 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.

$$R_O = \frac{\phi R_i - \gamma_D (DL) - \Sigma(\gamma \text{ (Other Effects)})}{\gamma_O} \qquad \text{(Equation 1)}$$

where:

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

2.3.1.1 *Rating Evaluations*

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

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

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.

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

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.

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

2.4 Health Monitoring Approach

2.4.1 Theory of this Approach

As outlined in section 1 of this report, Bridge 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. This stage 2 of the project 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.

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

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 that uses 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.

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

where:

FPE = Fitness for Purpose Evaluation
γ_o R_o = Ultimate Traffic Live Load Capacity
UTL Effect = Ultimate Traffic Load Effect derived from the health monitoring data

Generally a Fitness for Purpose evaluation greater than 100% indicates that the structure is "Fit for Purpose", while a 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 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 Big Wainihinihi 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 to identify critical locations for health monitoring instrumentation.

3.1 Bridge Description

The Big Wainihinihi River bridge is located on SH73 over the Big Wainihinihi River in Westland, near Otira. The bridge (Figure 3.1) consists of six simply supported spans (11.8 m), each with four steel girders (at 1000 mm centres), supporting a baulked timber deck. Construction of the 70 m-long bridge was completed in 1935.

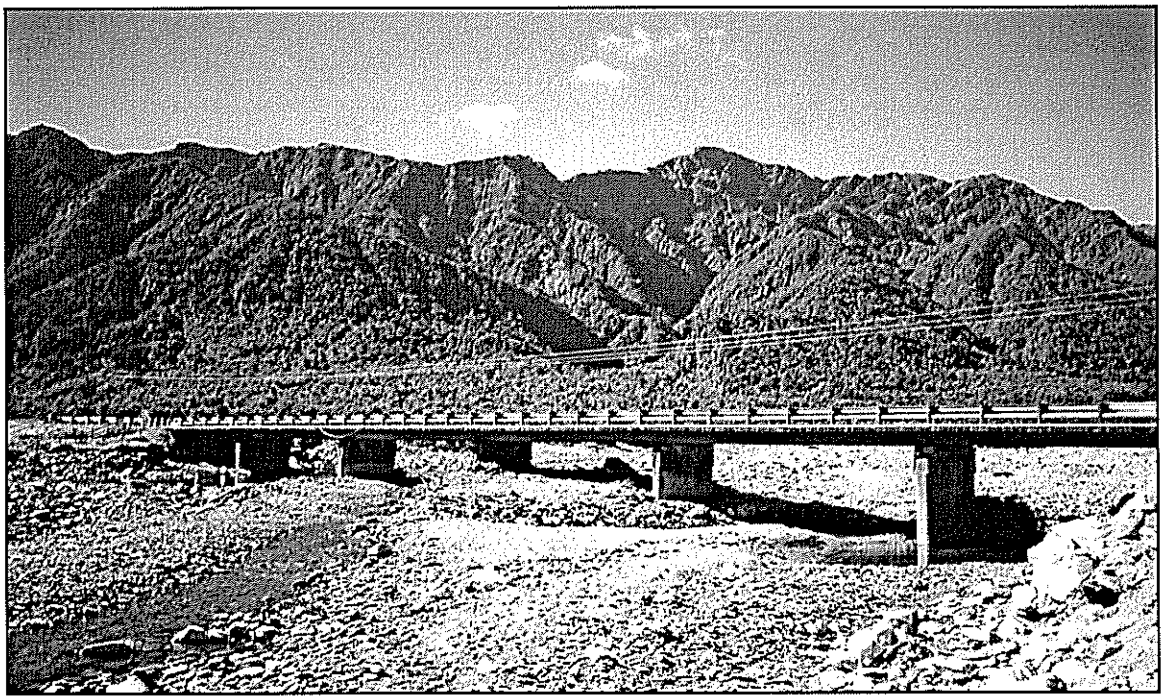


Figure 3.1 Big Wainihinihi River bridge, in Westland, South Island, New Zealand.

The speed limit over the bridge is 80 km/h but the bridge is only one lane in width (Figure 3.2) with a tight bend at its eastern end. This restricts the traffic to speeds typically less than 60 km/h. The width of the bridge and the road profile is illustrated in Figure 3.2, which also shows the path taken by vehicles using the bridge.

The deck of the structure consists of timber baulks with a chipseal wearing surface. During the testing of the structure the underside of the timber deck was noted to be covered with moss, indicating that moisture is present in the deck for extended periods. However, the condition of the timber based on a visual inspection was sound. Figure 3.3 shows the soffit of the structure illustrating that some moisture is in the timber deck.

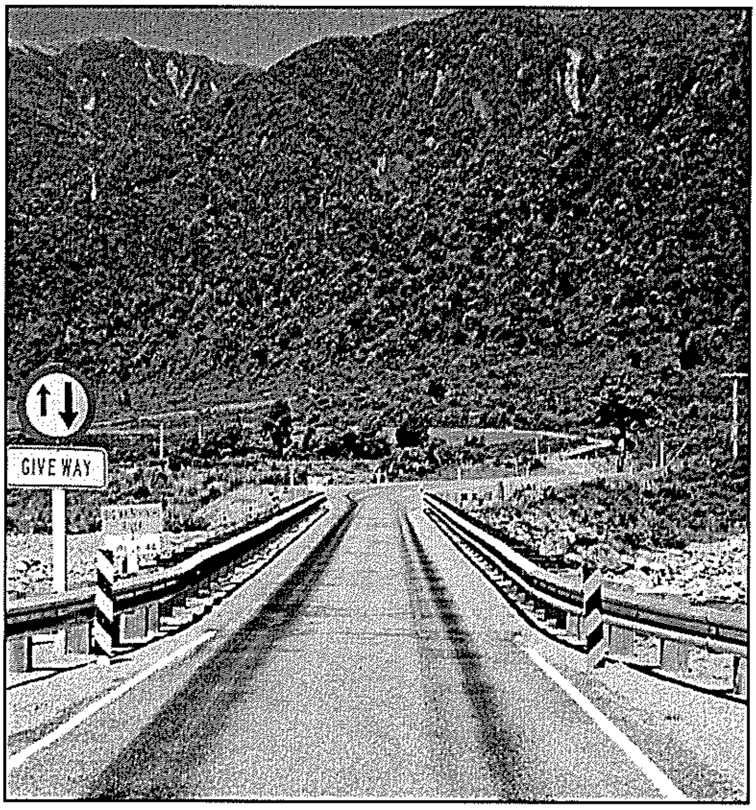


Figure 3.2 Vehicle path along the chipseal wearing surface of the bridge decking.

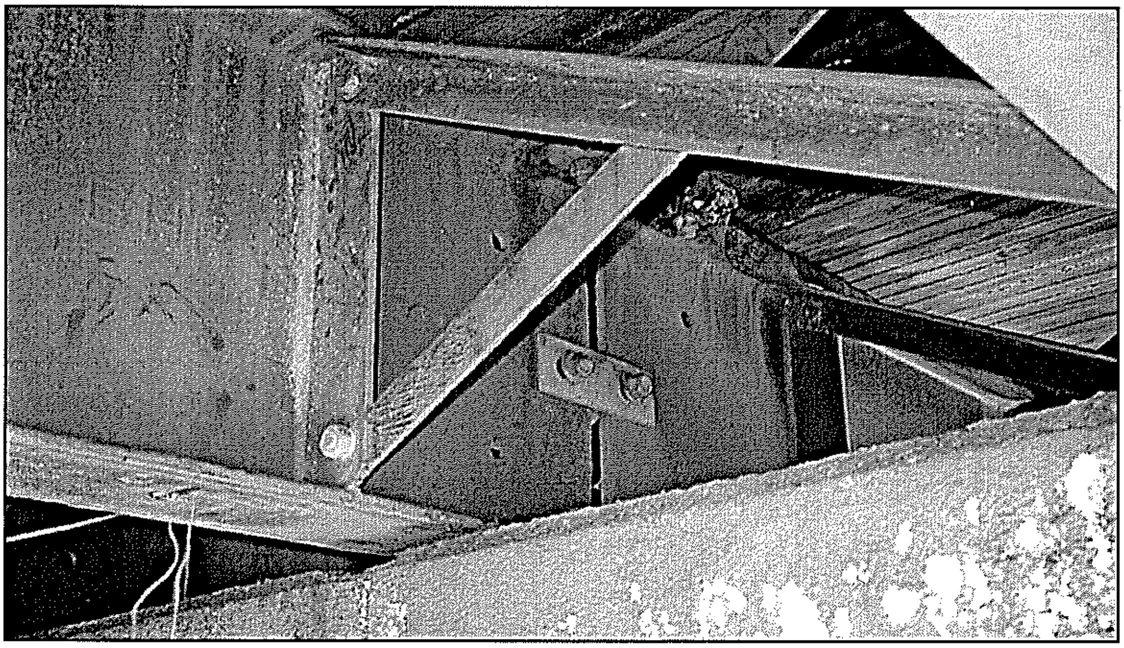


Figure 3.3 Soffit of the bridge superstructure and cross-bracing details.

The current rating of the bridge in the TNZ Structural Inventory is:

- Bridge Class 51%
- Deck Capacity Factor (DCF) 0.89

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

3.2 Structural Assessment

To identify the critical failure modes of the superstructure, an analysis of the structure was conducted using the 0.85 HN and 0.85 HO loading (see section 2.1 of this report). Results for the "known vehicle" 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 this bridge superstructure was investigated using a "grillage analysis"[1]. The dimensions of the structure were taken from the "as constructed" plans, which were confirmed by on-site measurements.

The material properties for the steel girders and the timber deck were not available. The properties used for the steel girders were obtained from Section 6.3.4 of the Bridge Manual, and the properties of the timber decking have been based on a conservative assumption of the timber deck grade being F8. The material properties used in the analysis of this bridge are listed below:

- Steel Girders - E = 200 000 MPa
- Steel Girders - f_y = 230 MPa
- Timber Deck - E = 9100 MPa
- Timber Deck - f'_b = 25 MPa (F8)

The loads applied to the grillage analysis included the 0.85 HO vehicle and the 0.85 HN vehicle.

3.2.1 Girder Bending

A summary of the maximum bending moments resulting from the various loads applied to the grillage model is presented in Table 3.1. The results in the table are not factored. The bending moments presented in the table represent the worst case effects with respect to vehicle eccentricity within the limits of the guardrail.

Table 3.1 shows that the 0.85 HO vehicle caused the maximum response. The maximum bending moment in the girders caused by the dead load is 104 kNm. The differences in the bending moments for each girder are related to the additional load effect of the guardrails and the material weight on the outside girders.

[1]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).

Load	Girder 1	Girder 2	Girder 3	Girder 4
Dead Load	104	73	73	104
Test Vehicle	155	125	112	79
0.85 HN Vehicle	170	146	134	105
0.85 HO Overload Vehicle	304	237	215	174

The bending capacity of the main steel girders of the superstructure was calculated in accordance with NZS:3404, Part1:1997 (SANZ 1997). A number of issues are related to calculating the girder bending capacity for this bridge, including:

1. *The effectiveness of the timber deck in providing restraint to the top flange.* The Bridge Manual allows timber decks to provide effective lateral restraint only if the deck fastenings are adequate in number and condition.

2. *The effectiveness of the diaphragms in preventing rotation of the beams.* The diaphragms consist of two diagonal members with no pin through the centre where the diagonals cross. There are no steel horizontal members at the top and bottom flanges at these locations. Therefore, unless the timber deck can provide lateral support, the diaphragms are not effective in preventing rotation of the beams.

For this analysis the assumption has been made that the deck does not provide full lateral restraint to the top flange. However it has been assumed that the deck, along with the diaphragms, can provide full lateral and torsional restraint at the diaphragm locations. Consequently the bending capacity of the girders has been calculated based on an effective girder length of 3.9 m (length between diaphragms). Consequently, the full plastic hinge capacity of the girders cannot be developed.

The nominal member moment capacity (ϕM_{bx}) based on this effective length (3.9 m) is 497 kNm. The bending strength of this bridge could be increased if the lateral restraint of the girders was increased sufficiently to allow the girders to develop the full plastic moment capacity.

3.2.2 Girder Shear

The shear force in each girder was also found using the grillage analysis. The results are presented in Table 3.2. The shear capacity (ϕV_{vu}) of the main girders, found in accordance with NZS:3404, Part1:1997, is 684 kN. This is well above the design shear force presented in Table 3.2. Therefore, shear strength does not govern the load rating of the structure.

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

Load	Girder 1	Girder 2	Girder 3	Girder 4
Dead Load	32	25	25	32
Test Vehicle	53	39	44	25
0.85 HN Vehicle	57	48	52	33
0.85 HO Overload Vehicle	121	78	87	54

3.2.3 Deck Bending

The critical load case for bending in the deck was determined using the Deck Rating 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 (Bridge Manual, Section 3.1.2). The HN tyre load caused the greatest effects because of the load concentration. The bending moment caused by this wheel load was 10.7 kNm. The bending strength (ϕM) of the decking (based on the properties listed in section 3.2 of this report), was 21.5 kNm. This was based on the width of deck used to support the wheel load, as in the recommendations in the Bridge Manual. The calculation for the timber deck capacity was based on AS1720.1:1997 Timber Structures Code (SAA 1997).

3.3 Theoretical Load Rating

The process required to determine the theoretical load rating of a bridge using the Bridge Manual is outlined in section 2.2 of this report. The results of the load rating of the structure are presented in Table 3.3. The rating has been assessed for the bending and shear in the girders and the bending in the deck. The table also presents a comparison of the load rating calculated by Infratech Systems & Services, and the load rating in the current TNZ Structural Inventory. A value of 1.3 was used for the impact factor in calculating the load ratings.

Table 3.3 Summary of theoretical ratings 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) (%)
Girder Bending	497 kNm	304 kNm	170 kNm	104	59	84	51
Girder Shear	684 kN	121 kN	57 kN	32	273	454	–
Deck Bending (F8)	21.5 kNm	–	9 kNm	–	–	1.3	0.89

The overall rating of the girders is taken as the minimum value calculated from the girder actions evaluated (bending and shear).

For this bridge, the rating is the minimum of the ratings based on shear and bending, which is 59% (HO Infratech rating), and the critical failure mode is the midspan bending of the girders. This result compares well to the rating of 51% which is documented in the TNZ Structural Inventory (1999).

There is a difference between the rating calculated by Infratech for the deck (DCF), and that quoted by the TNZ Structural Inventory. It is expected that this is related to differences in assumptions regarding material properties between the Australian Timber Code (SAA 1997) and the Bridge Manual.

Because the posting evaluation (i.e. 84%) is less than 100 %, the normal practice would be to post this bridge with a load limit. No evidence of posting was found while on site and it is understood that this bridge is not currently posted.

3.4 Summary

The Big Wainihinihi River bridge, in Westland, was analysed using a grillage analysis to determine the bending moment and shear in the girders of a typical span, based on various vehicle loadings.

The superstructure rating was 59% (Infratech) compared with 51% (TNZ Structural Inventory), indicating that the actual capacity of the bridge is substantially below that required by the current Bridge Manual. The Infratech rating of the deck was 1.30 suggesting that it is adequate, but the TNZ Structural Inventory rating of the deck is 0.89, suggesting some cause for concern. The reason for the difference in deck ratings appears to be related to differences in assumed timber properties, but this issue may warrant further investigation.

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

4. Health Monitoring Programme

The programme of Health Monitoring on the Big Wainihinihi River bridge involved two components:

- Short-term health monitoring of the ambient heavy vehicle traffic for a period of approximately 3 days.
- Testing using a heavy vehicle of known mass and dimensions (i.e. the known vehicle) to compare with the health monitoring data.

The details and results of the programme are presented in this section of the report.

4.1 Instrumentation

The instrumentation installed on the bridge was positioned so that it could record the bridge response to the ambient heavy vehicle traffic and the known vehicle at the critical sections for bending. Table 4.1 summarises the objectives for the transducer locations.

Table 4.1 Summary of objectives for each transducer.

Transducer	Position	Required Results
S(3-1)	Midspan Span 3, Girder 1, bottom flange	Midspan bending strain, lateral load distribution
S(3-2)	Midspan Span 3, Girder 2, bottom flange	Midspan bending strain, lateral load distribution
S(3-3)	Midspan Span 3, Girder 3, bottom flange	Midspan bending strain, lateral load distribution
S(3-4)	Midspan Span 3, Girder 4, bottom flange	Midspan bending strain, lateral load distribution
S(3-3A)	Span 3, bottom flange of Girder 3, 300 mm from centre of bearing pad (western Greymouth end)	Support conditions
S(3-3B)	Span 3, bottom flange of Girder 3, 300 mm from centre of bearing pad (eastern Otira end)	Support conditions
S(3-3T)	Midspan Span 3, Girder 3, top flange	Degree of composite action between girder and deck
SD(3)	Span 3, bottom of deck, between Girders 3 & 4	Deck bending due to wheel loads

The instrumentation used on the bridge included seven Foil Strain Gauge transducers installed on the main girders and one Demountable Strain Gauge transducer fitted to the underside of the timber deck. The locations of these transducers are illustrated on the plan in Figure 4.1. An "S" represents a strain measurement in the main girders, and an "SD" indicates strain measurement in the deck. The spans were numbered from the western (Greymouth) end of the bridge.

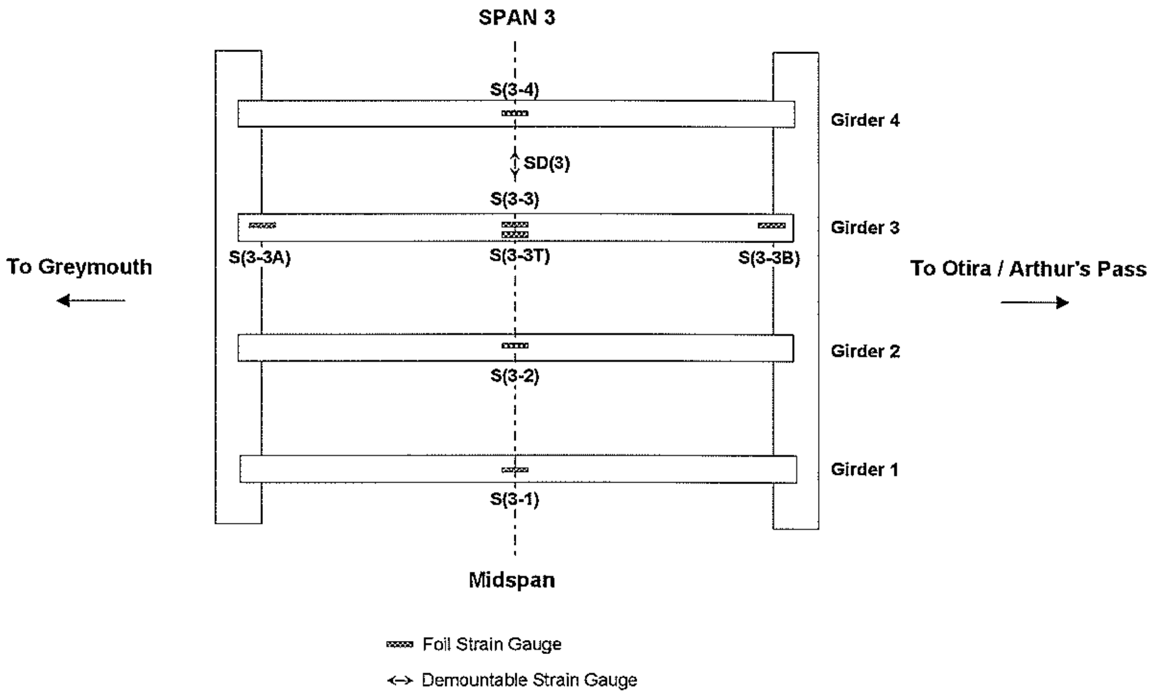


Figure 4.1 Instrumentation plan for the bridge.
S; SD = Strain measurement in main girder, and deck, respectively.

Transducer S(3-3T) was located on the top flange of Girder 3 as shown in Figure 4.2. All other foil strain gauge transducers were fitted to the soffit of the bottom flange of the respective girders.

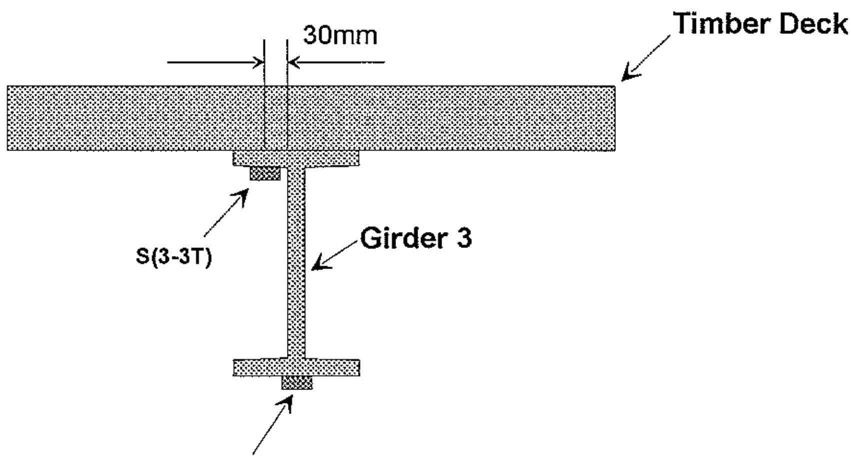


Figure 4.2 Position of transducer S(3-3T) (not to scale).

Transducers S(3-3A) and S(3-3B) were also fitted to the bottom flange at either end of Girder 3. The exact locations are illustrated on the girder elevation in Figure 4.3. These strain gauge transducers were installed to investigate the support conditions for the girders.

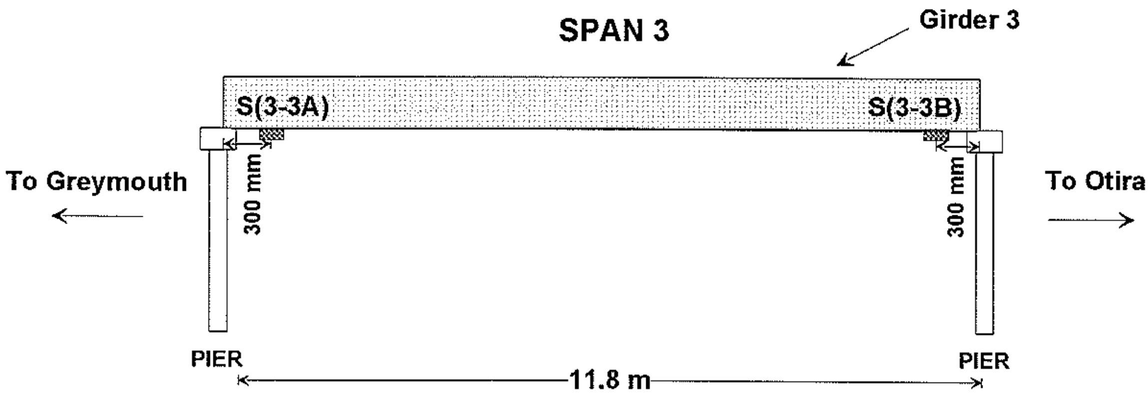


Figure 4.3 Position of transducers S(3-3A) & S(3-3B).

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

4.2 Health Monitoring Programme

The health monitoring of the structure began on Thursday 26 November 1998 and continued until Sunday 29 November, giving a total monitoring period of approximately 67 hours. The monitoring period was extended from the typical 24-hour period because of the low volume of heavy vehicles using the bridge. During the 3-day monitoring period, the response of the bridge to 152 heavy vehicles was recorded (excluding the passage of the known vehicle).

The "known vehicle" was a 7-axled heavy vehicle of known mass and dimensions. It was used in the component of testing that was performed to provide a comparison of the health monitoring data against a known load.

This testing was conducted on Friday 27 November 1998. The vehicle used for the testing was supplied by T. Croft Ltd, and is shown in Figure 4.4. The axle weights and configuration are illustrated in Figure 4.5.

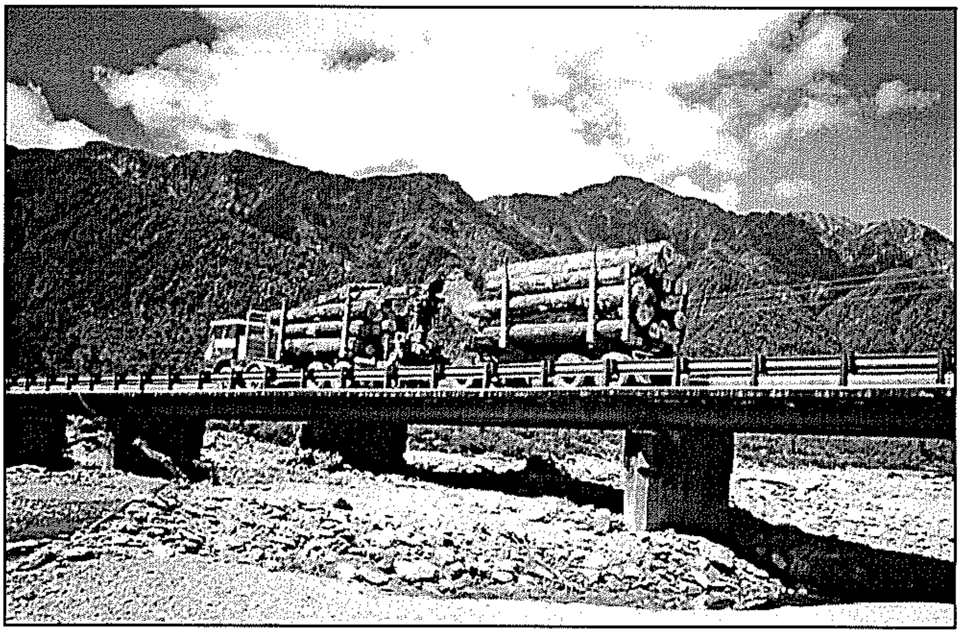


Figure 4.4 The known vehicle used for behavioural testing.

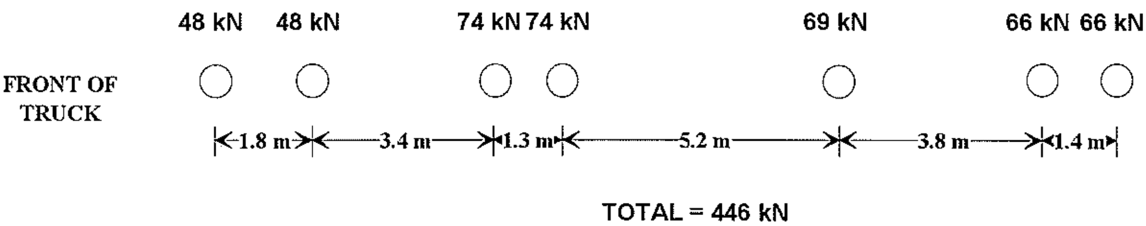


Figure 4.5 Axle mass and configuration of the known vehicle.

The testing with the known vehicle was conducted by recording the response of the bridge to the vehicle as it passed over the bridge at different speeds. The tests were conducted with the vehicle travelling in both directions (east and west) at a crawl (10 km/h), 20 km/h, 40 km/h, 60 km/h and 80 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 them for a few minutes at a time. This ensured minimal traffic interruptions and also allowed the continuous monitoring of heavy vehicles between test runs with the known vehicle.

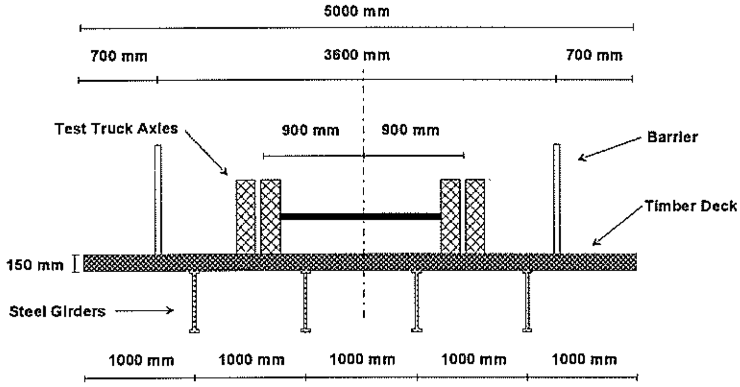


Figure 4.6 Lateral position of known vehicle during testing.

Figure 4.7 shows the known vehicle on the bridge and the small amount of lateral clearance for the vehicle on each side. This restricts the path of the vehicle which limits the variation in the lateral distribution of load.

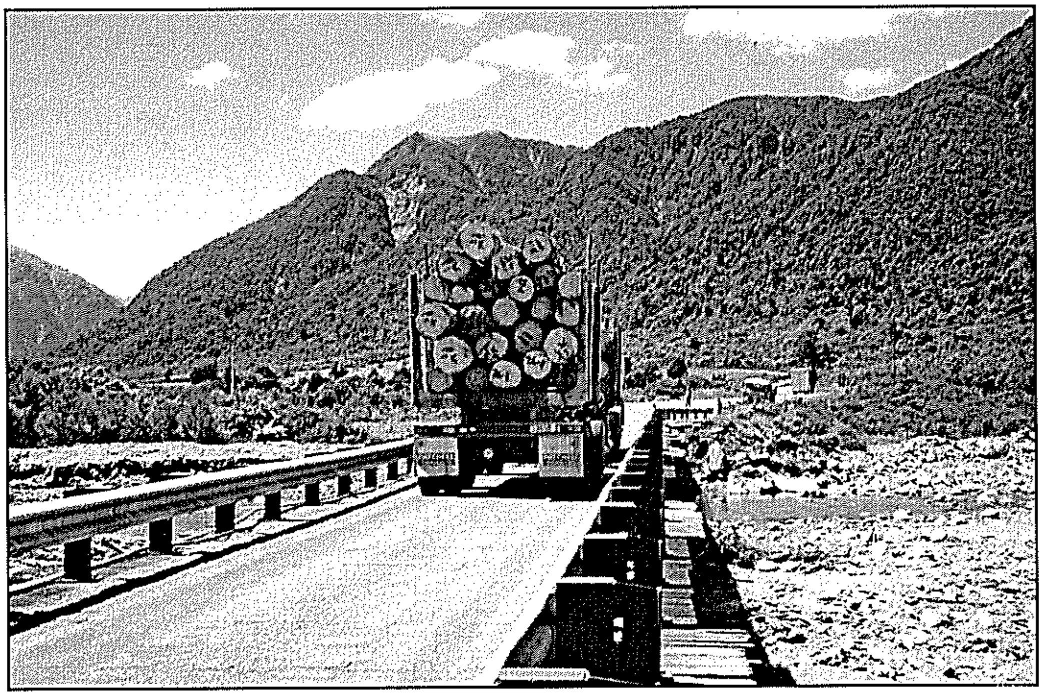


Figure 4.7 Known vehicle on the bridge during testing.

4.3 Short-Term Health Monitoring Results

A health monitoring period of 3 days was chosen for the bridge, based on previous experience (section 2.4 of this report), and the relatively low heavy vehicle numbers using the route.

4.3.1 Girder Response

A typical strain response versus time was graphed (as waveform) for the midspan bending strains recorded during the health monitoring for the passage of a heavy vehicle. It is presented in Figure 4.8. This response suggests that the distribution of load is fairly uniform between girders and that the bridge is not well damped because the free vibrations continued for a long time.

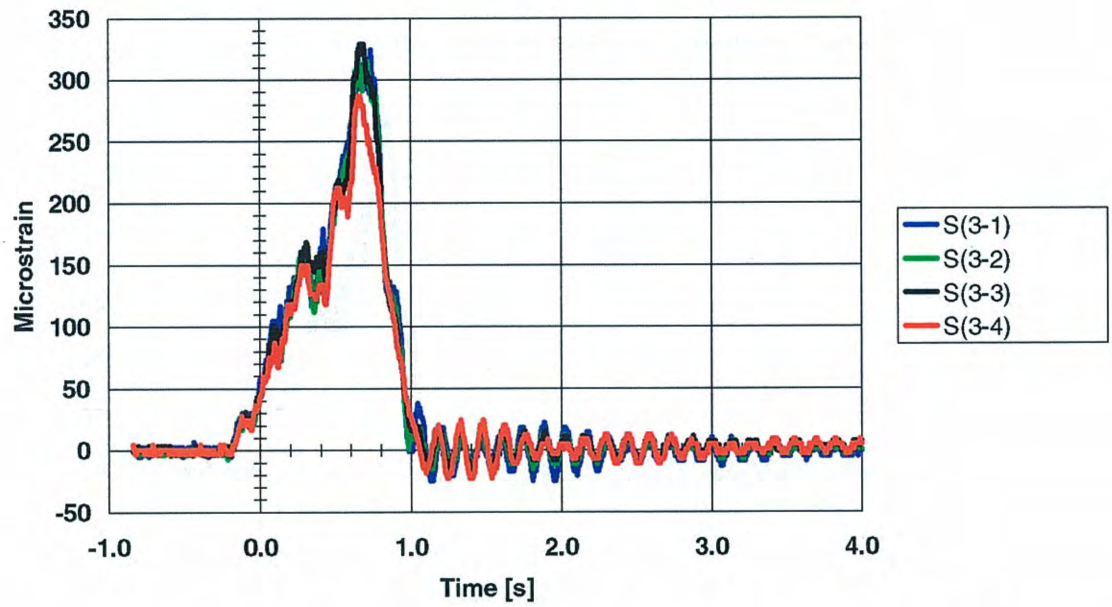


Figure 4.8 Strain response versus time for midspan transducers of Span 3, for event recorded on 28 Nov 1998 at 14:03.

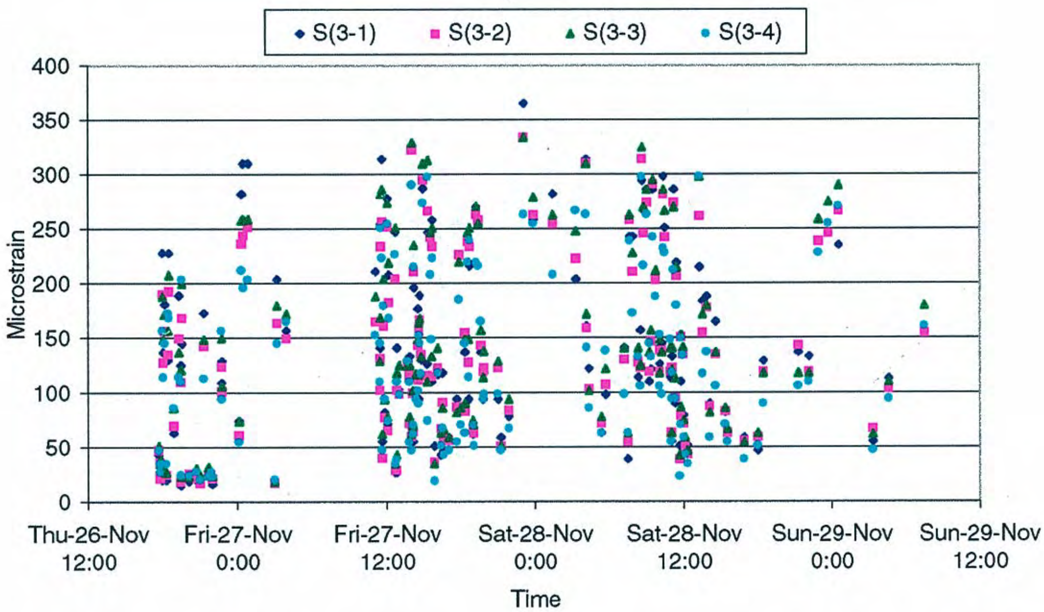


Figure 4.9 Scatter diagram for midspan transducers of Span 3 for monitoring period.

The scatter diagrams represent the maximum strains recorded during the passage of each heavy vehicle (including the known vehicle). Figure 4.9 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 waveforms for the remaining transducers on Girder 3 are illustrated in Figure 4.10. The plot shows the tensile (i.e. positive) response of transducers S(3-3A) and S(3-3B), both located on the bottom flange of Girder 3, and the compressive (i.e. negative) response of transducer S(3-3T) located on the top flange of Girder 3. The scatter diagram for these transducers is presented as Figure 4.11.

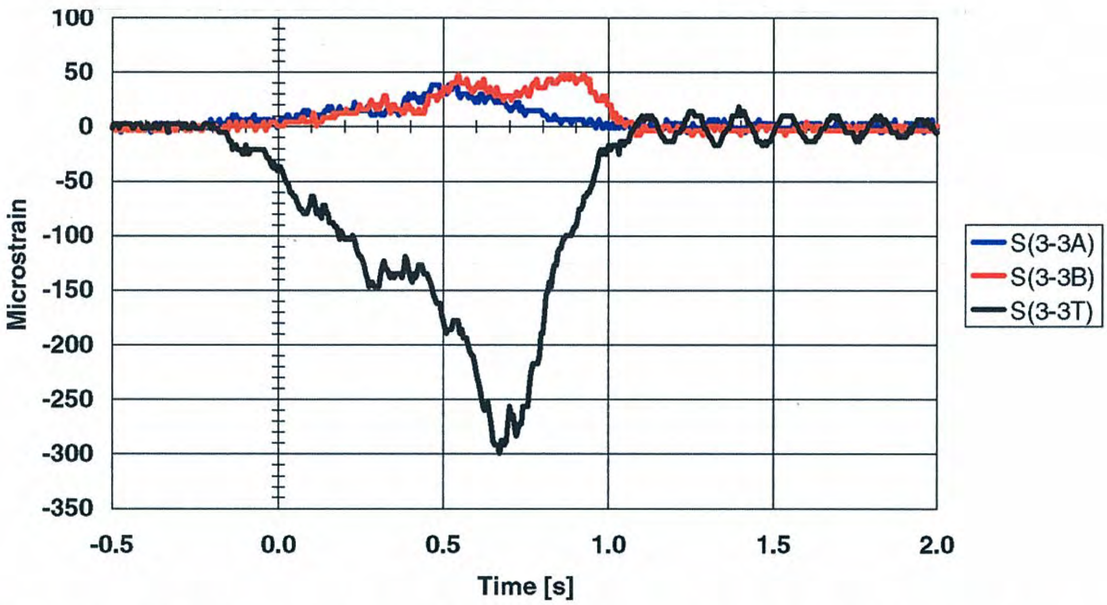


Figure 4.10 Strain response versus time for Girder 3 transducers S(3-3A), S(3-3B) & S(3-3T).

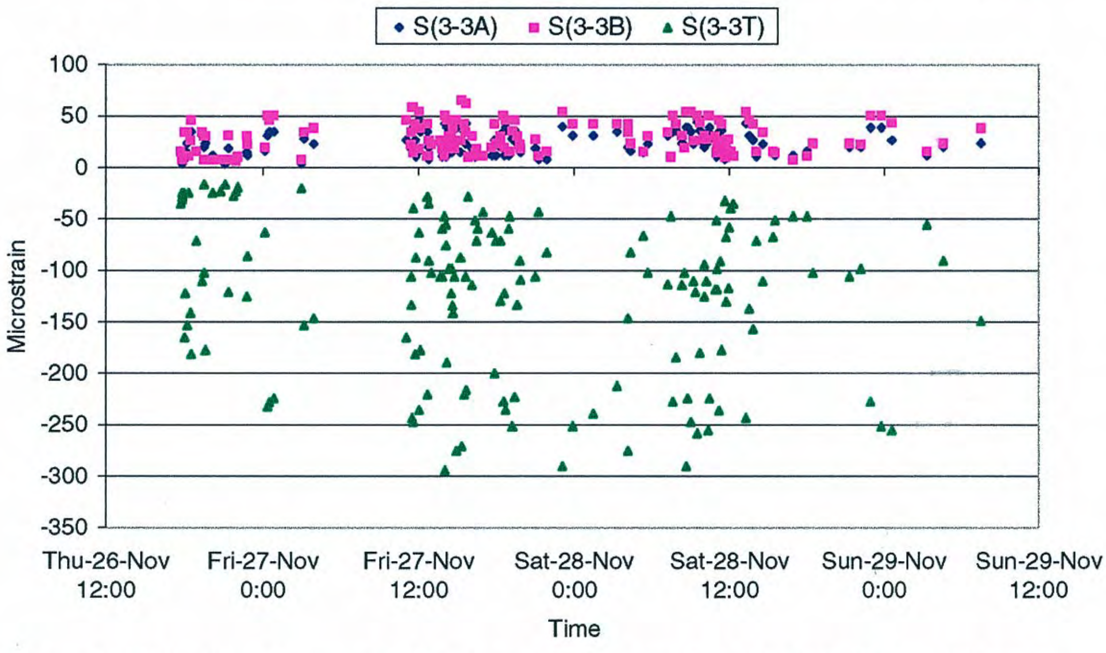


Figure 4.11 Scatter diagram for Girder 3 transducers S(3-3A), S(3-3B) and S(3-3T).

The data from the scatter diagram can also be plotted on a histogram that incorporates a cumulative distribution. An example is presented for transducer S(3-1) in Figure 4.12.

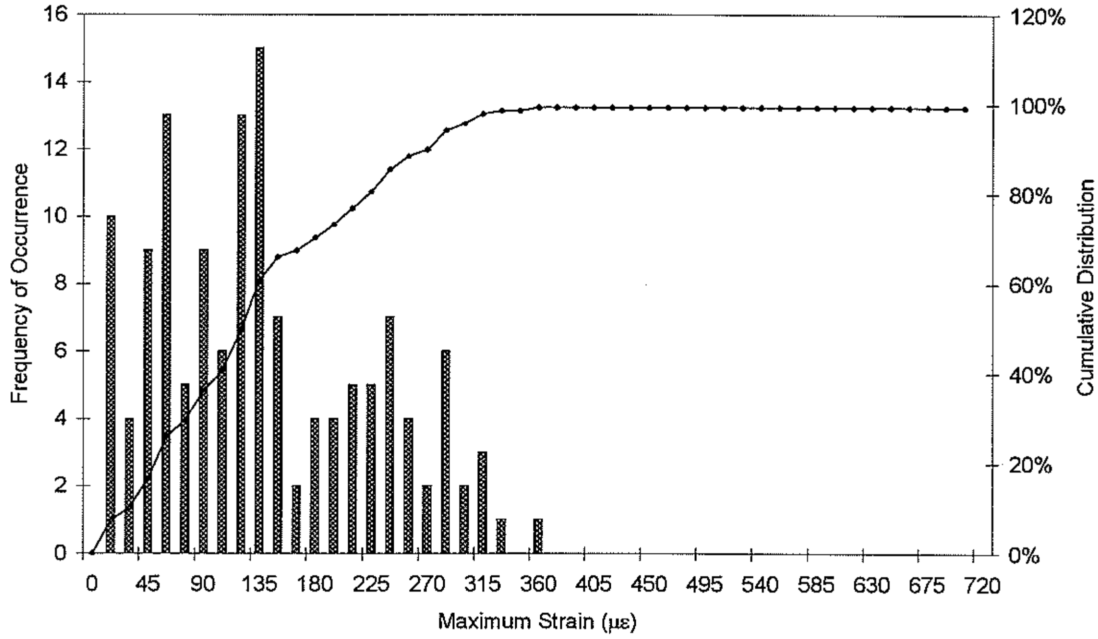


Figure 4.12 Histogram and cumulative distribution function for midspan transducer S(3-1).

The cumulative distribution function can then be plotted on a probability scale known as an "inverse normal scale". The inverse normal plot for each of the midspan transducers is presented in Figure 4.13. 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 which is outlined in section 2.4 of this report.

The maximum results along with the extrapolated results for the midspan transducers and all transducers on Girder 3 are presented in Table 4.2. The results for the deck transducer are presented in section 4.3.2 of this report.

The inverse normal distributions for the remaining transducers on Girder 3 are presented in Figure 4.14.

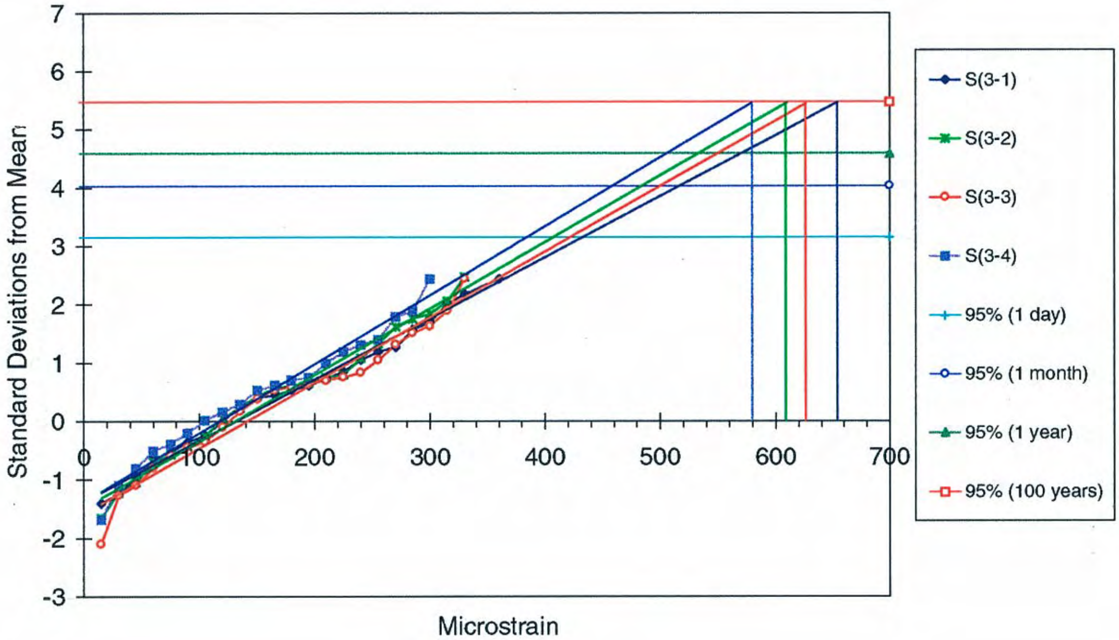


Figure 4.13 Inverse normal plot for midspan transducers (S(3-1), S(3-2), S(3-3), S(3-4).
(see Table 4.2 for results of plots)

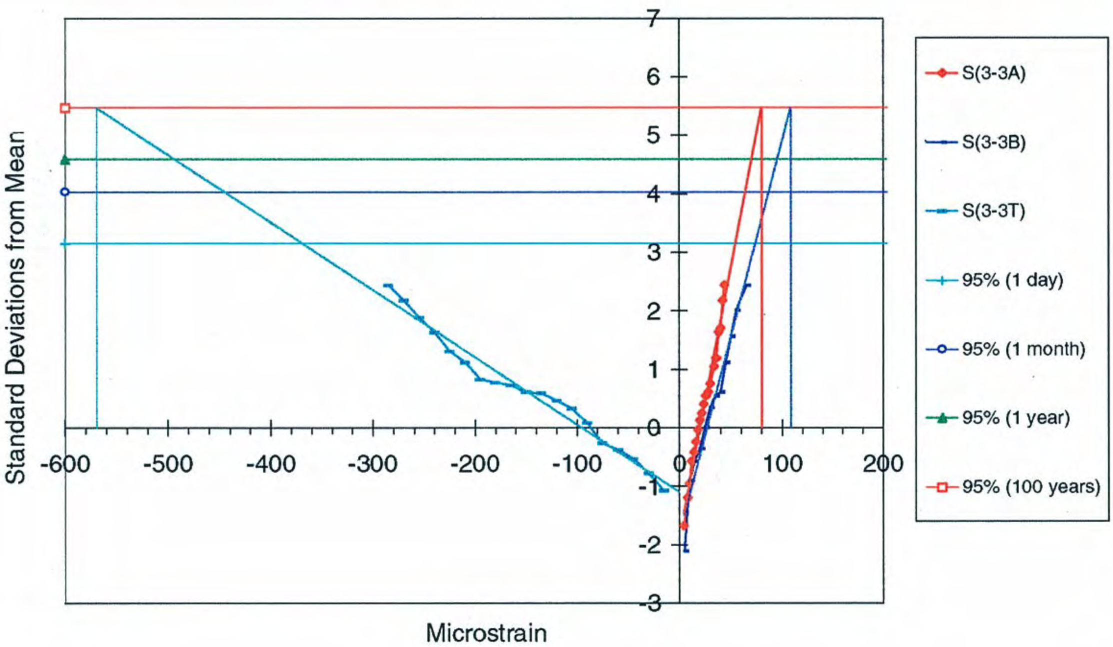


Figure 4.14 Inverse normal distribution for Girder 3 transducers (S(3-3A), S(3-3B), and S(3-3T).
(see Table 4.2 for results of plots)

Table 4.2 Extrapolated data obtained from inverse normal distribution.
(obtained from Figures 4.13 and 4.14)

Transducer	Maximum Recorded Value (Monitoring)	Extrapolated Value (95% confidence limit) for 1 year	Extrapolated Value (95% confidence limit) for 100 years
S(3-1)	365	570	660
S(3-2)	334	540	620
S(3-3)	334	560	640
S(3-4)	298	520	590
S(3-3A)	44	70	80
S(3-3B)	66	100	110
S(3-3T)	–294	–490	–560

Composite action and support conditions

The detailed drawings of the structure show that the support types result in simply supported girders. Transducers S(3-3A) and S(3-3B) were installed on Girder 3 to determine if the restraint conditions were influencing the girder performance. The recorded data indicate that the beams are behaving as simply supported members, and that the support conditions are not influencing the performance of the girders.

The presence of any composite action between the steel girders and the timber deck was determined by recording the response of the transducers on the top flange and bottom flange of Girder 3. Figure 4.15 presents the comparison of the top and bottom flange strains recorded from the health monitoring. A ratio of 1:1 of top to bottom strains shows that no composite action is present.

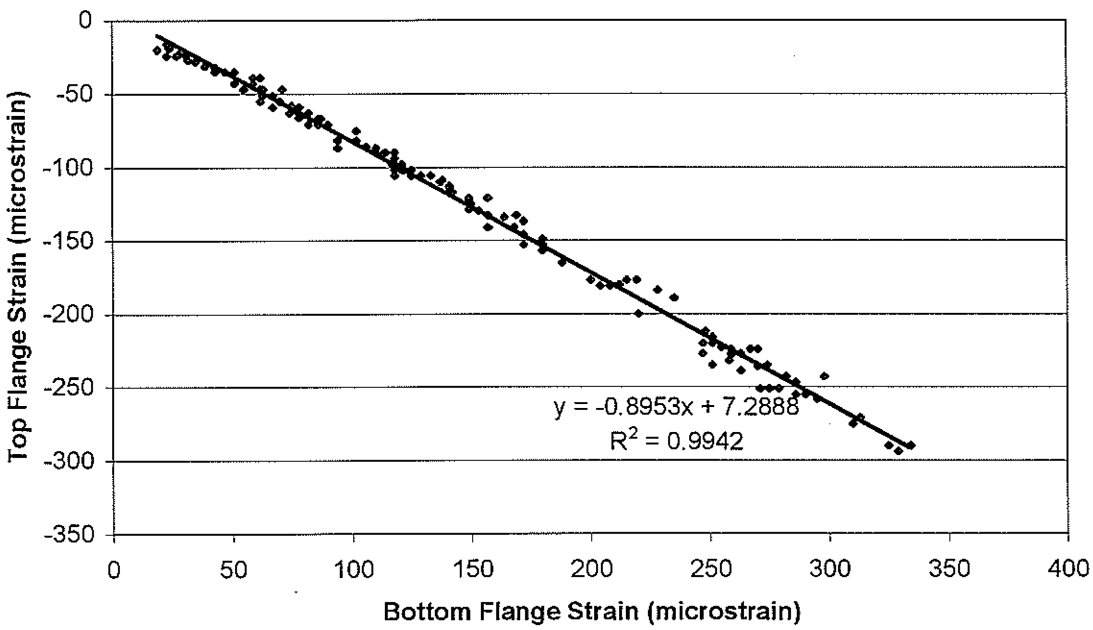


Figure 4.15 Relationship between top and bottom flange strains, for transducers S(3-3T) and S(3-3) respectively, on Span 3, Girder 3.

Figure 4.15 shows that the relationship between the two transducers is approximately 0.9:1. Transducer S(3-3) was located at the soffit of the girder (i.e. bottom flange) while transducer S(3-3T) was installed on the underside of the top flange (Figure 4.2, Table 4.1). When the relationship was adjusted to account for the differences in position, it was found that no composite action was occurring between the girders and the deck.

4.3.2 Deck Response

Figure 4.16 presents the response of the deck for the same event that is shown in Figure 4.8. The transverse bending strains in the deck respond to each individual wheel. Thus the 6 response peaks suggest that the vehicle was a six-axle truck travelling at approximately 60 km/h.

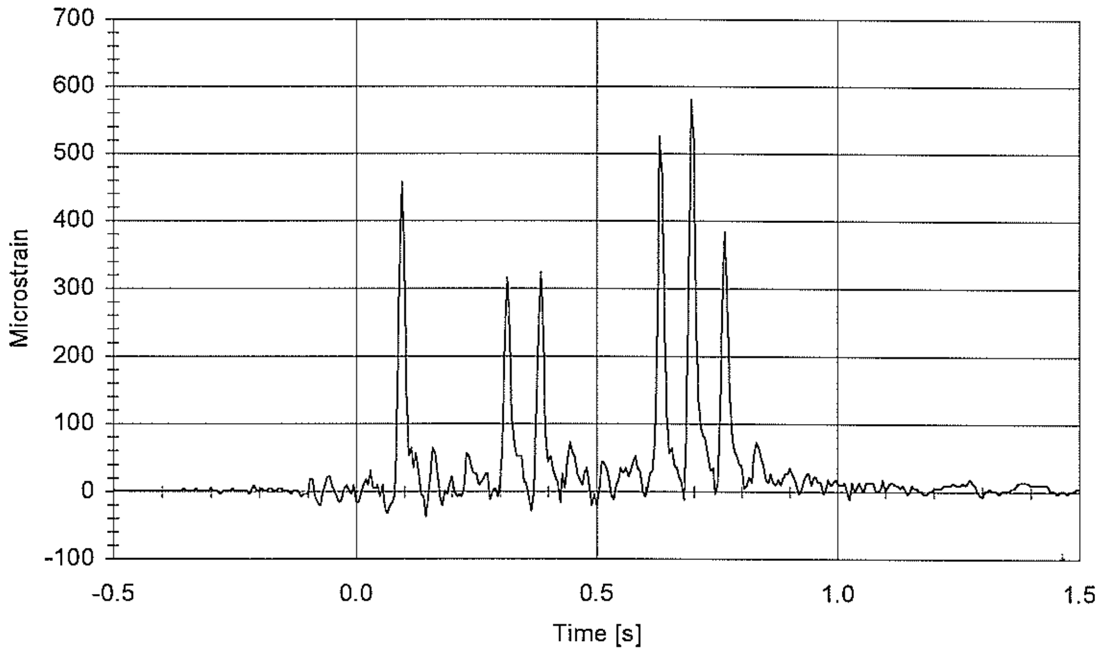


Figure 4.16 Deck strain versus time, for event recorded on 28 Nov 1998 at 14.03.

Figure 4.17 displays the scatter diagram for the deck transducer. The maximum strain event recorded for this transducer was 576 $\mu\varepsilon$. This maximum is approximately 120 $\mu\varepsilon$ greater than the next largest event, indicating either a more heavily loaded axle, or the wheel was positioned in the centre of the deck to give the highest bending moment.

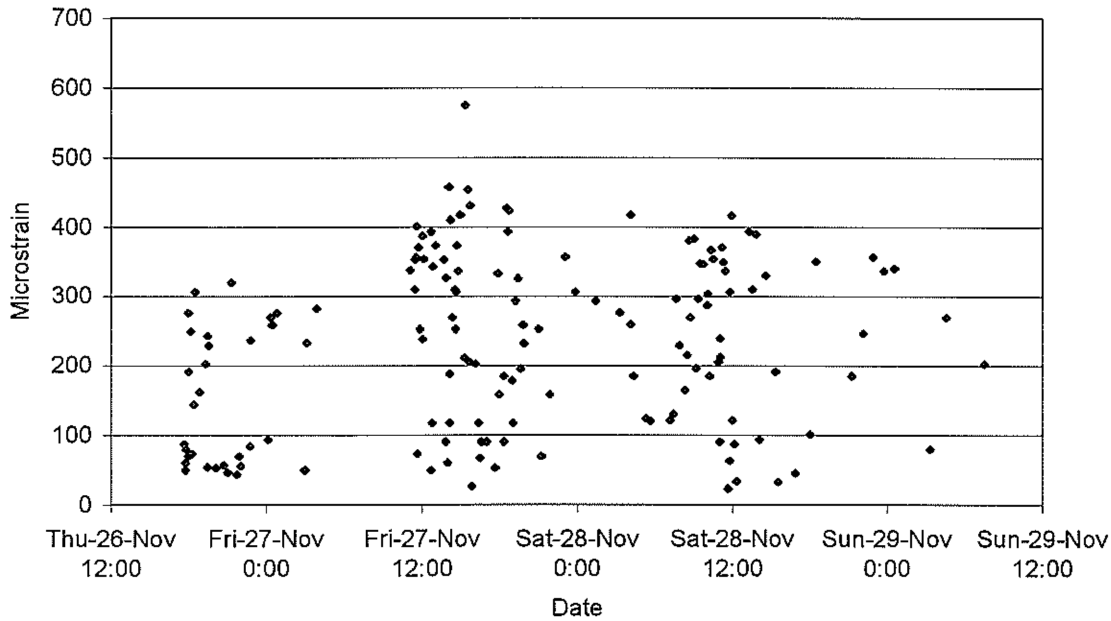


Figure 4.17 Scatter diagram for deck transducer SD(3).

The inverse normal plot for the deck transducer is presented in Figure 4.18. The tail (extreme top) of the curve shows a "kink" and a point with a magnitude of about 576 $\mu\varepsilon$. This point represents the outlier on the scatter diagram that was shown in Figure 4.17. The uneven distribution is caused primarily by the large variation in the magnitude of the events. This is probably caused by the sensitivity of the deck response to the wheel position. A longer monitoring period may result in a more linear distribution.

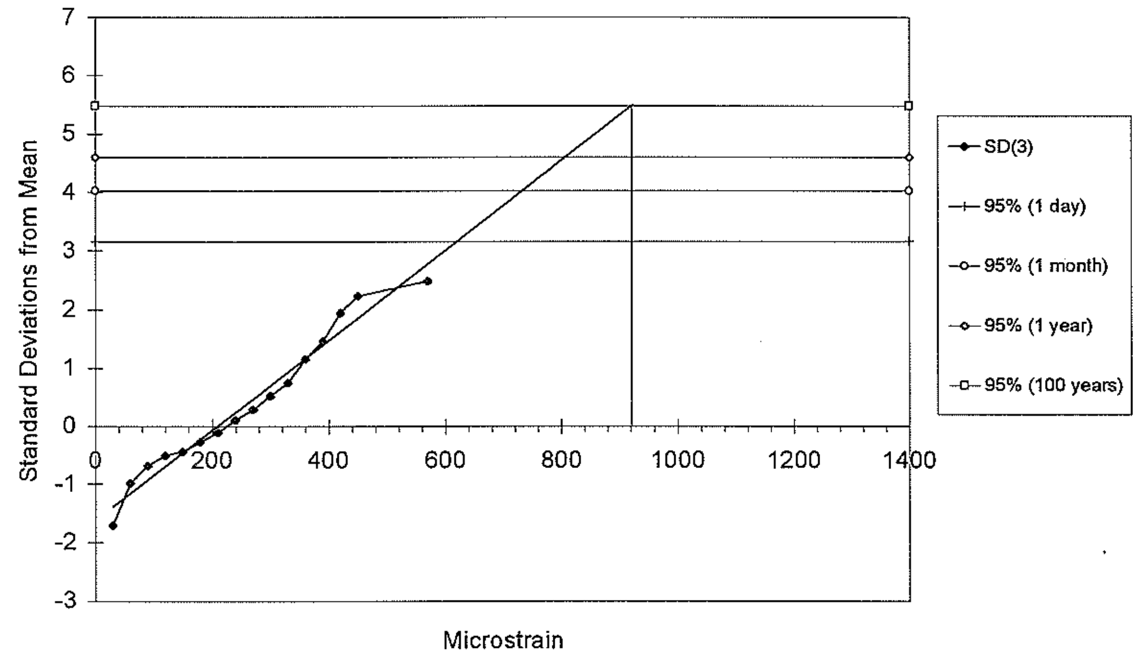


Figure 4.18 Inverse of the normal distribution function for deck transducer SD(3).

The extrapolated values for the deck transducer taken from the inverse normal plot are 807 $\mu\varepsilon$ and 920 $\mu\varepsilon$ for the 95% confidence limit for 1 year and 100 years respectively.

4.4 Known Vehicle Testing

4.4.1 Girder Response

A typical waveform for the passage of the known vehicle is presented in Figure 4.19. In each case tensile strains are positive and compressive strains are negative. Despite the range of magnitudes, all waveforms showed similar responses including that from the transducer S(3-3T) which was located at the top of the flange and displayed compression results.

The transducers showed unforced response after the vehicle had passed over the span. Such behaviour represents free vibration of the girders after the truck has passed over the structure. The figure shows the oscillation of the girders continuing for approximately 3 seconds after the truck has moved off Span 3.

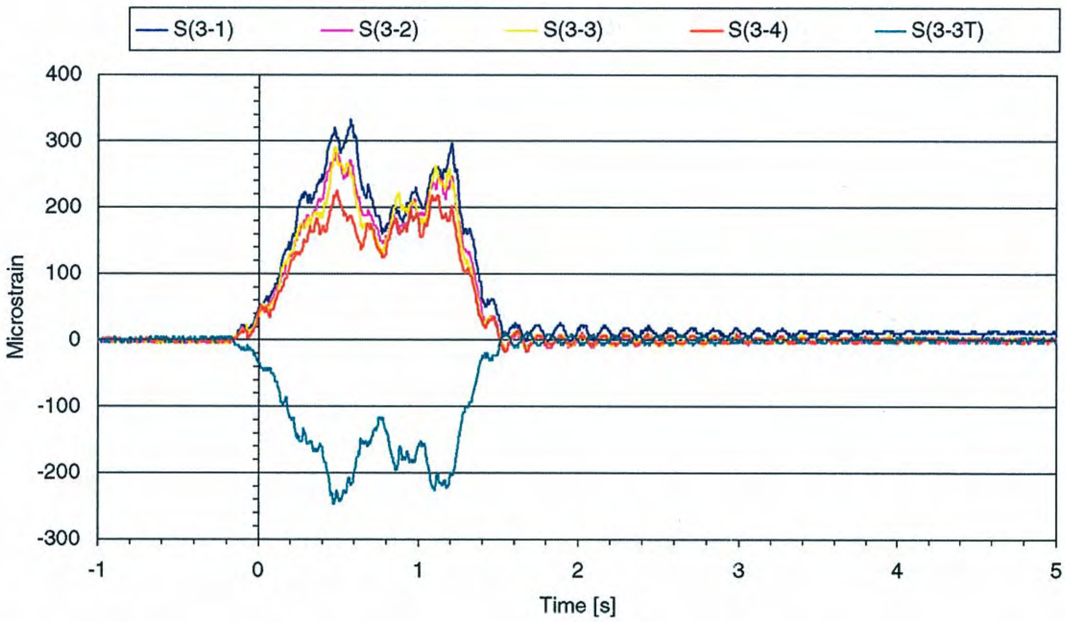


Figure 4.19 Waveform for midspan transducers on Girder 3, obtained from the known vehicle travelling at 60 km/h in an easterly direction.

The maximum strains for each transducer recorded during the testing with the known vehicle are presented in Table 4.3. The table does not include the results for the deck transducer.

Table 4.3 Maximum recorded strains for known vehicle testing.

Transducer	Maximum Strain (με)
S(3-1)	337
S(3-2)	294
S(3-3)	333
S(3-4)	318
S(3-3A)	43
S(3-3B)	74
S(3-3T)	–262

4.4.2 Load Distribution

The distribution of load into each girder is presented in Figure 4.20. The distribution shows the largest variation of load into the outside girders (1 and 4) by a maximum of approximately 8%, while the load into Girders 2 and 3 varied by only 2%. While the data for the test conducted at 20 km/h (east) were corrupted, it is unlikely that the results would be affected by the lack of this data.

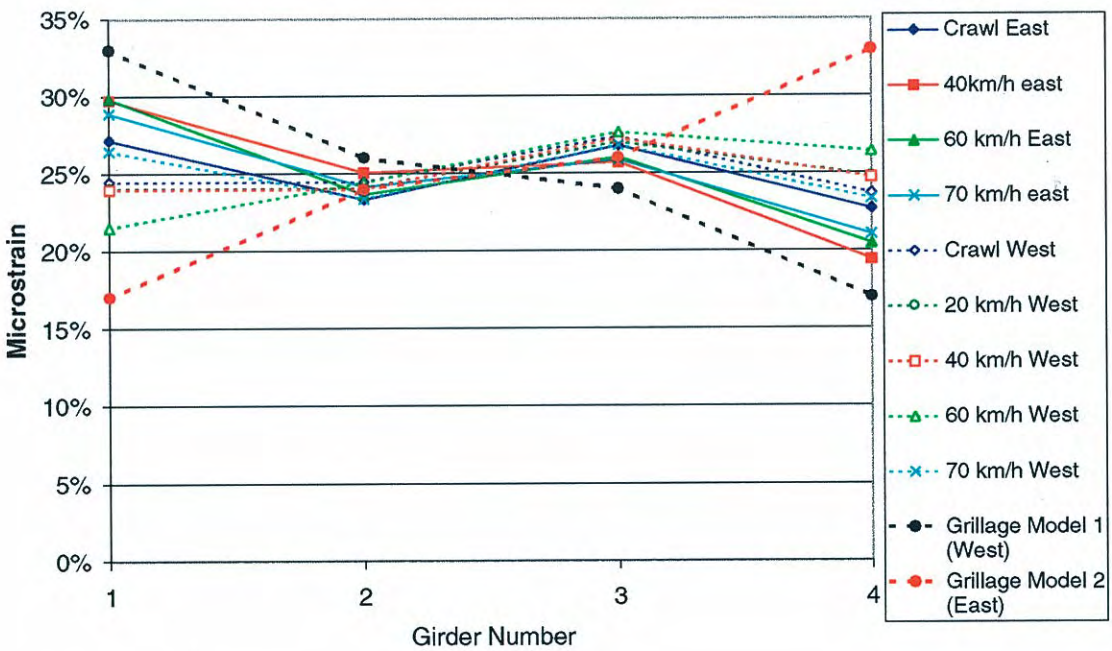


Figure 4.20 Distribution factors for known vehicle testing (at various vehicle speeds and directions).

The distributions presented in Figure 4.20 are consistent, except for the test performed at 60km/h, travelling west. This distribution shows more load into Girder 4 and less load into Girder 1. This was probably because the truck had a different lateral position compared with the other tests.

Figure 4.20 also presents the theoretical load distribution from the two grillage analyses (from west and east) of the known vehicle load. These analyses were performed with the known vehicle positioned at the greatest allowable eccentricity (600 mm from either kerb). The results show that there is less variation in the distribution from the actual vehicles compared with the grillage results.

4.4.3 Dynamic Increment and Natural Frequency

The dynamic response of the main girders in the structure is clearly evident from the waveforms presented in Figure 4.19. The waveform shows the response of the girder after the known vehicle has passed over Span 3.

The natural frequency based on the data recorded from the known vehicle testing is 7.4 Hz. The damping ranges between 0.8% and 1.5%. This damping is low and reflects the prolonged oscillation of the bridge after a vehicle has passed over the instrumented span, as discussed in section 4.4.1 of this report.

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

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

The response of the crawl test was used for the static result in the calculation of dynamic increment. The data are plotted for each of the midspan transducers in Figure 4.21, which presents the dynamic increment data for the known vehicle travelling east, and Figure 4.22, which presents the data for the known vehicle travelling west.

The dynamic increment data in Figures 4.21 and 4.22 show varied responses for each direction. The dynamic increment for the vehicle travelling east is the greatest for Girders 1 and 2 (i.e. S(3-1) and S(3-2)). However, when the vehicle is travelling west, the dynamic increment is the greatest for Girders 3 and 4 on the opposite side of the bridge. This is caused by variations in the lateral positioning of the known vehicle for each pass.

Despite the differences in the magnitude of the dynamic increment, the maximum value should be considered. The maximum dynamic increment recorded was 27% for Girder 4. Based on this, an appropriate Impact Factor for this bridge would be 1.27. This compares with a value of 1.3 recommended by the Bridge Manual.

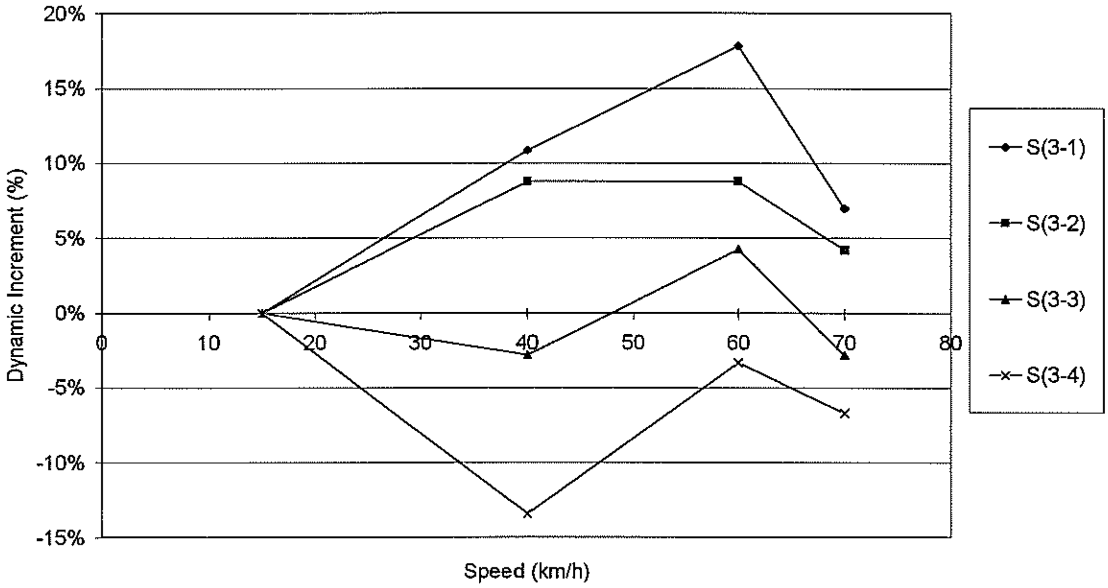


Figure 4.21 Dynamic increment (%) versus speed (km/h) for the known vehicle travelling east, for Girders 1 to 4.

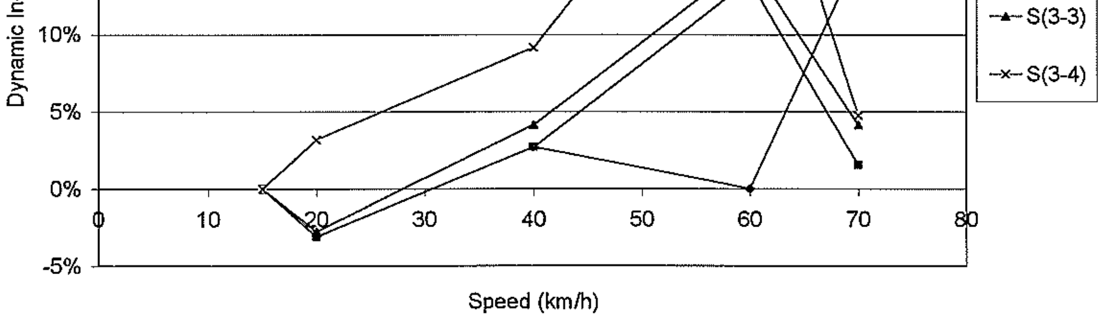


Figure 4.22 Dynamic increment (%) versus speed (km/h) for the known vehicle travelling west, for Girders 1 to 4.

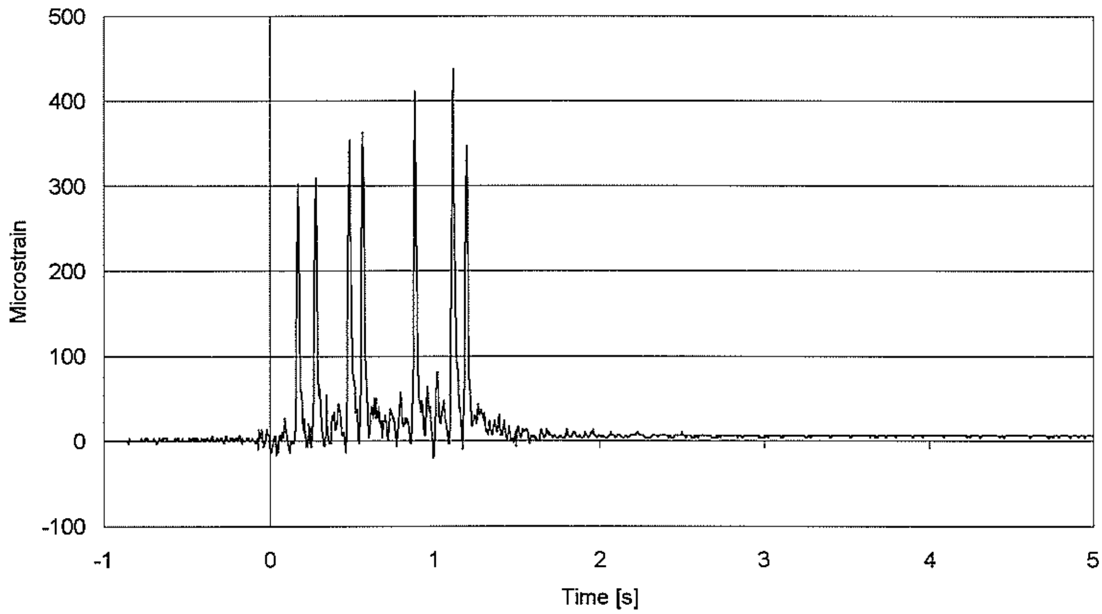


Figure 4.23 Waveform for the deck transducer SD(3) obtained from the known vehicle travelling east at 60 km/h.

Table 4.4 Summary of health monitoring data.

Transducer	Maximum Recorded Value (Test Vehicle)	Maximum Recorded Value (Health Monitoring)	Extrapolated Value 95% Confidence limit for 1 year	Extrapolated Value 95% Confidence limit for 100 years
S(3-1)	337	365	570	660
S(3-2)	274	334	540	620
S(3-3)	333	334	560	640
S(3-4)	318	298	520	590
S(3-3A)	47	47	70	80
S(3-3B)	74	66	100	110
S(3-3T)	–282	–294	–490	-560
SD(3)	482	576	810	920

4.4.4 Deck Response

Figure 4.23 illustrates the response of the deck transducer SD(3) as the known vehicle passes over the midspan of Span 3. The waveform shows seven distinctive "spikes" which represents each of the seven axles as they pass over the deck. The maximum response presented by this waveform is 465 $\mu\varepsilon$. The waveform also shows very little vibration after the passage of the vehicle. The dynamic increment of the deck was reviewed and a maximum value of 10% was determined based on the test results.

4.5 Summary

A summary of the data recorded for the health monitoring and the testing with the known vehicle is presented in Table 4.4. In most cases the results obtained for the maximum response of the structure to the ambient heavy traffic were larger than the response to the known vehicle. This suggests that the level of overloading for the heavy vehicles on this route is well controlled. Strain results from Girder 4 indicate that the response of this girder to the known vehicle was 20 $\mu\varepsilon$ higher than the largest recorded health monitoring event. This is probably the result of the varied lateral positioning of the known vehicle during the testing.

The comparison of the health monitoring results with the results from the known vehicle testing proves that the ambient heavy vehicle traffic sample is significant and appropriate for use in the extrapolation of the ultimate traffic load effect.

5. Fitness for Purpose Evaluation

5.1 Steel Girders

The structural analysis assessment described in section 3.2 of this report indicated that midspan bending was the critical mode of failure for the structure. The Fitness for Purpose has been determined based on this failure mode. The moment capacity for the ultimate traffic live load was 351 kNm ($\phi M_{bx} - 1.4DL$) and corresponds to a strain in the soffit of the girder of 703 $\mu\varepsilon$. Figure 5.1 illustrates the extrapolated health monitoring data with this ultimate live load strain (703 $\mu\varepsilon$) superimposed on the graph. This information shows that at the ultimate traffic live load strain (95% in 100 years) is approaching the capacity limit (i.e. 703 $\mu\varepsilon$) and that Girder 1 is the most critical.

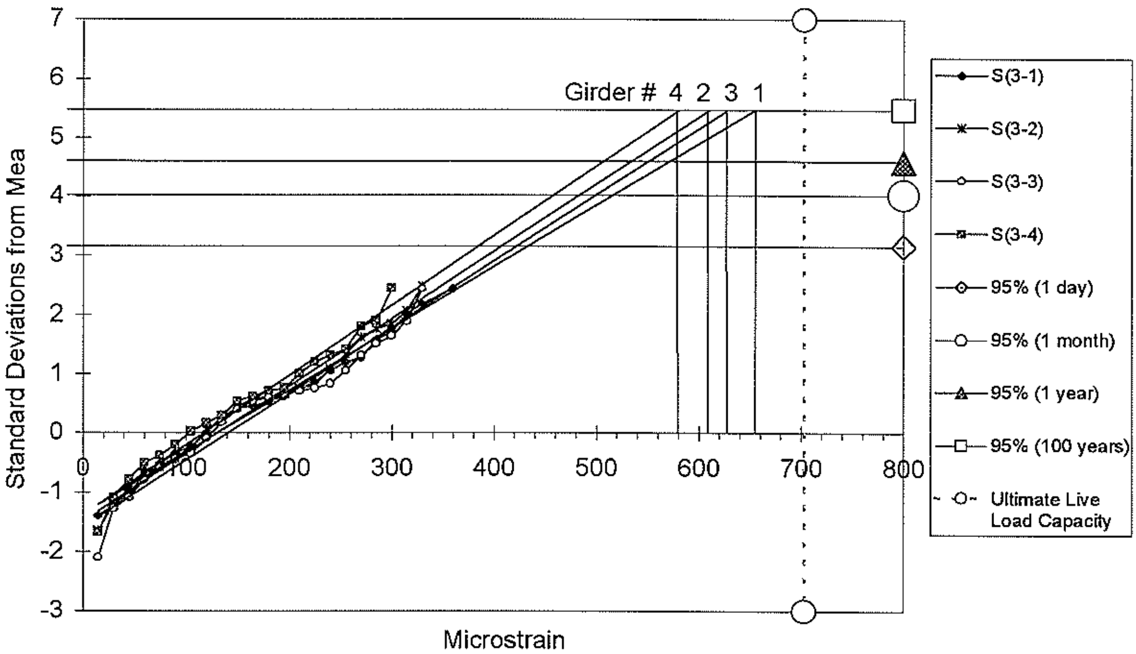


Figure 5.1 Inverse normal distribution plots for midspan transducers showing the ultimate live load capacity.

Table 5.1 summarises the calculation of the Fitness for Purpose evaluation, based on these data. The method for the calculation of this evaluation was outlined in section 2.4 (Equation 6) of this report, and involves dividing the ultimate traffic 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 107 %, with the edge girders being more critical than the middle girders.

This rating compares to the theoretical rating calculated for the 0.85 HO loading (59%) and for the 0.85 HN loading (84%) (Table 3.3). Comparison with the 0.85 HN loading is the most appropriate as this loading is related to ambient heavy vehicle traffic.

Table 5.1 Summary of Fitness for Purpose rating.

Item	Edge Girder	Middle Girder
Strength (ϕM_{bx})	497 kNm	497 kNm
Dead Load	104 kNm	73 kNm
Ultimate Live Load Capacity Moment ($\gamma_o R_o$)	351 kNm	395 kNm
Ultimate Live Load Capacity Strain ($\gamma_o R_o$)	703 $\mu\varepsilon$	790 $\mu\varepsilon$
Maximum Recorded Strain (Ambient Traffic)	365 $\mu\varepsilon$	334 $\mu\varepsilon$
Test Vehicle Strain	337 $\mu\varepsilon$	333 $\mu\varepsilon$
Ultimate Traffic Load Effect	655 $\mu\varepsilon$	641 $\mu\varepsilon$
Fitness For Purpose Rating	107 %	123 %

A summary of these results is presented on the capacity diagram in Figure 5.2, based on the results from transducer S(3-1) (on Girder 1) which is the most critical. The overall rectangle represents the ultimate capacity in bending (814 $\mu\varepsilon$). The diagonally hatched section on the figure represents the factored (equal to 1.4) dead load component (291 $\mu\varepsilon$). (See also section 3.2.1 of this report.)

The white unshaded area in Figure 5.2 is the Fitness for Purpose capacity of the structure available at the ultimate limit-state live loads (703 $\mu\varepsilon$). The darkened rectangles represent the magnitude of the recorded strains and the extrapolated strains from the health monitoring and testing with the known vehicle.

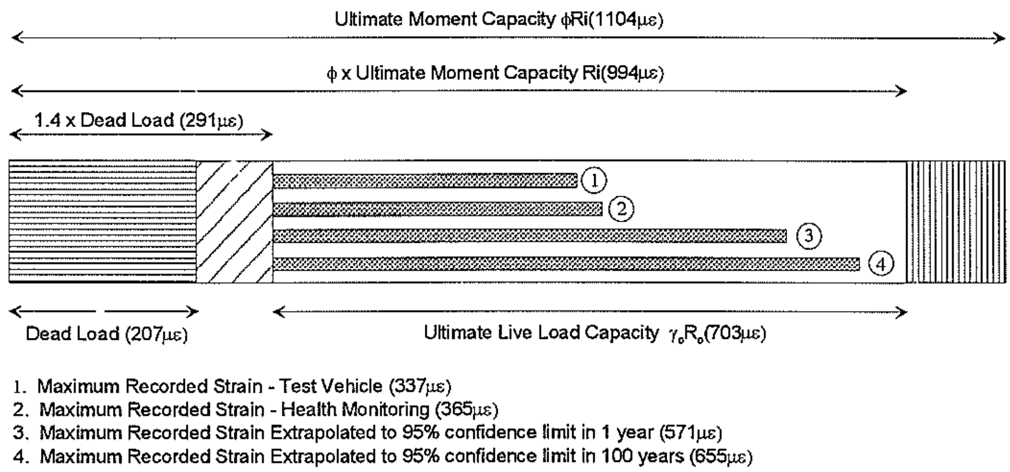


Figure 5.2 Summary of Fitness for Purpose based on limit-state design principles.

5.2 Timber Deck

The health monitoring data for the response of the deck to the ambient heavy vehicle traffic is presented in section 4.3.2 of this report. The ultimate (100 years) traffic live load strain extrapolated for the deck is 920 $\mu\varepsilon$. The failure strain for Grade F8 timber is approximately 1900 $\mu\varepsilon$. This gives a DCF of 2.0. This is much higher than the theoretical rating and probably reflects the difference between the wheel load used for the rating, and the actual wheel loads. Obviously the strains recorded in the deck are also very sensitive to wheel position.

The health monitoring of the deck indicates that the deck performance is satisfactory. However its condition should be checked regularly to check for deterioration, as this affects bridge capacity. The assumptions regarding the properties of the timber deck also need to be confirmed.

5.3 Summary

The Fitness for Purpose rating for this bridge, based on midspan bending of the main girders, is 107%. This rating is 23% higher (based on bridge class, derived by 107-84) than the 0.85 HN posting evaluation calculated in section 3.3 of this report. The structural action of this bridge is relatively simple and the health monitoring indicates that the girders are acting as a simply supported structure. Therefore it is expected that the reasons for the higher rating are principally related to the loading on the structure. Reasons for the higher rating obtained from the health monitoring include:

- The ambient heavy vehicle traffic is typically inducing lower bending moments than the 0.85 HN vehicle loading for this span. The known vehicle induced bending effects that were similar to that of the 0.85 HN vehicle loading and a significant proportion of the ambient heavy vehicle traffic induced effects that were less than this vehicle.
- The level of overloading on this bridge is not excessive.
- The 0.85 HN rating is based on the load being positioned on the grillage model close to the guardrail fence according to criteria in the Bridge Manual. The health monitoring data indicate that the distribution of load into these outer girders is not as great as this analysis suggests, because most of the vehicles travel down the centre of the bridge.

As outlined in section 3.2.1 of this report, the restraint of the girders was an issue affecting the ultimate bending capacity of girders. This report is based on the assumption that the girders are fully restrained at the diaphragms. However, for this restraint to be possible, the timber deck has to provide restraint to the top flange at these locations. The ability of the deck to provide this restraint depends on the connections between the deck and the girders. The results of the health monitoring indicate that, if the restraint of the girders is secure, the bridge is Fit for Purpose. However if this restraint was not present, then the risk of failure for this bridge would be unacceptable.

6. Conclusions & Recommendations

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

Efficient health monitoring requires placement of instrumentation in the most appropriate locations (those having greatest influence on bridge capacity or performance). Structural analysis of the bridge was used to identify these locations before setting up the site.

Midspan bending of the main girders and the performance of the deck were the governing factors affecting the capacity of the bridge. Subsequently the lateral restraint of the main girders was also identified as an important factor affecting the bending capacity of the main bridge girders.

6.1 Theoretical Results

Midspan bending of the main girders and the performance of the deck are the critical issues associated with the performance of the bridge (based on theoretical analysis). The Health Monitoring programme focussed on assessing the performance of these components.

The interpretation of the restraint of the girders at the diaphragms is a third critical issue with the girder capacity.

The theoretical assessment of the superstructure of the bridge found that the 0.85 HO rating evaluation was 59%, the 0.85 HN posting evaluation was 84%, and the DCF was 1.3 for the deck. These values compare with the ratings from the TNZ Structural Inventory of 51% for the HO rating and 0.89 for the DCF. According to normal practice this bridge should be posted, with its load limits based on this assessment. However, posting does not appear to have occurred.

6.2 Health Monitoring Results

The Health Monitoring programme found that:

- The girders are acting as simply supported members and the timber deck is not acting compositely with the girders.
- The ambient heavy vehicle traffic is typically inducing lower bending moments than the 0.85 HN vehicle loading for this span.

Consequently the level of overloading on this bridge is well controlled, and in addition, the number of heavy vehicles is low.

- Because the vehicles generally drive down the centre of the bridge, the distribution of load into the edge girders is not as excessive as the theoretical analysis.

- The impact factor of 1.3 obtained from the Bridge Manual is appropriate for this bridge.

6.3 Fitness for Purpose Evaluation

The Fitness for Purpose rating for this bridge based on the critical midspan bending of the main girders is 107%. This rating indicates that the bridge is safely carrying the heavy vehicle traffic currently using this route. However it should still be posted.

The Fitness for Purpose evaluation for the deck was 2.0 which is satisfactory, but its condition should be checked regularly for deterioration, and the assumptions regarding the material properties for the deck should be reviewed.

6.4 Recommendations

The conventional rating procedures based on the Bridge Manual indicate that this bridge should be posted with a load limit. However, the results of health monitoring (Fitness for Purpose evaluation) indicate that posting is not necessary. As the bridge is not currently posted, a decision on whether or not the bridge should be posted is therefore required.

The Fitness for Purpose evaluation was based on the assumption that the main steel girders are laterally braced at the third point bracing locations. The true nature of the restraint provided to the girders by the decking system is critical to girder capacity, but is unclear. Consequently the recommendation is that the restraint of the girders should be reviewed. In this context the following should be noted:

- If adequate (third point) restraint is not present, the capacity of the bridge would be significantly reduced below that used in the Fitness for Purpose evaluation;

- If restraint is significantly better than that assumed (third point), then the bridge capacity could be substantially increased above that used in the Fitness for Purpose evaluation.

The two recommended approaches to resolve the issue of lateral restraint of the girders are:

- Test the bridge to determine the lateral restraint characteristics: this would involve some additional health monitoring, and the use of a known heavy vehicle;

- Stiffen the third point restraints of the girders to ensure that they provide effective restraint: this could involve adding horizontal bracing members to the girder bottom flanges at diaphragm locations.

7. References

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