Health Monitoring of Superstructures of New Zealand Road Bridges:

Bealey (Waimakariri) Bridge, Canterbury

Transfund New Zealand Research Report No. 166

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Bealey (Waimakariri) Bridge, Canterbury

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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 Bealey Bridge over the Waimakariri River, on State Highway 73 east of Arthur's Pass, Canterbury Region, South Island, was built in 1935. It was selected as one of these ten bridges because it is an aging single-lane, reinforced-concrete structure with a low conventional strength rating. It also has some flexural cracking in the main girders.

The 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. The performance of the substructure and the deck have not been considered in this assessment. The critical parameters considered were midspan bending strength and shear strength of the main concrete girders.

Theoretical Analysis
The theoretical analysis of the superstructure of the bridge carried out by Infratech Systems & Services gave the 0.85 HO* rating as 56% and the 0.85 HN* posting as 66%. These compare favourably with the value listed in the 1999 Transit New Zealand Structural Inventory of 65% for the 0.85 HO rating. According to normal practice, this bridge should be posted using this assessment. The Deck Capacity Factor (DCF) in the 1999 Transit New Zealand Inventory is 0.89. However deck capacity was not monitored as part of this investigation.

Health Monitoring Results
The results of the Health Monitoring programme show:

- Some continuity exists between spans, indicating some bearing restraint effects.
- Typical heavy vehicles are inducing bending moments in the bridge that are up to 90% of the 0.85 HN posting load. However a significant portion of the heavy vehicles using this route are lighter than these vehicles, and also traffic volumes are low.
- Because of the narrow width of the structure, the lateral position of the heavy vehicles is restricted and thus distribution of load in to each girder is consistent.
- An impact factor of 1.07 was found to be appropriate for this bridge. This is much lower than the impact factor of 1.30 used to determine the load rating as detailed in the Transit New Zealand 1994 Bridge Manual. The impact factor is low because the road alignment and width restrict the vehicle speed.

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* HO Highway overweight vehicles; HN Highway normal vehicles

Fitness for Purpose Evaluation

- The Fitness for Purpose Evaluation for this bridge, based on the critical midspan bending of the main girders, was 125%. This rating indicates that the bridge is safely carrying the heavy vehicle traffic currently using this route.

- The higher Fitness for Purpose Evaluation, compared with the posting evaluation, is mainly related to the lightly laden heavy vehicle traffic using this route and to the low impact factor.

Recommendation

The theoretical assessment of this bridge suggests that it should be posted with a load limit, although Health Monitoring results suggest that the bridge is performing adequately and that posting is not necessary.

Traffic characteristics may have changed significantly since this data was gathered (in particular as a result of the opening of the Otira viaduct), and thus the Fitness for Purpose Evaluation may require revision. Consequently the recommendation is to review the traffic characteristics.

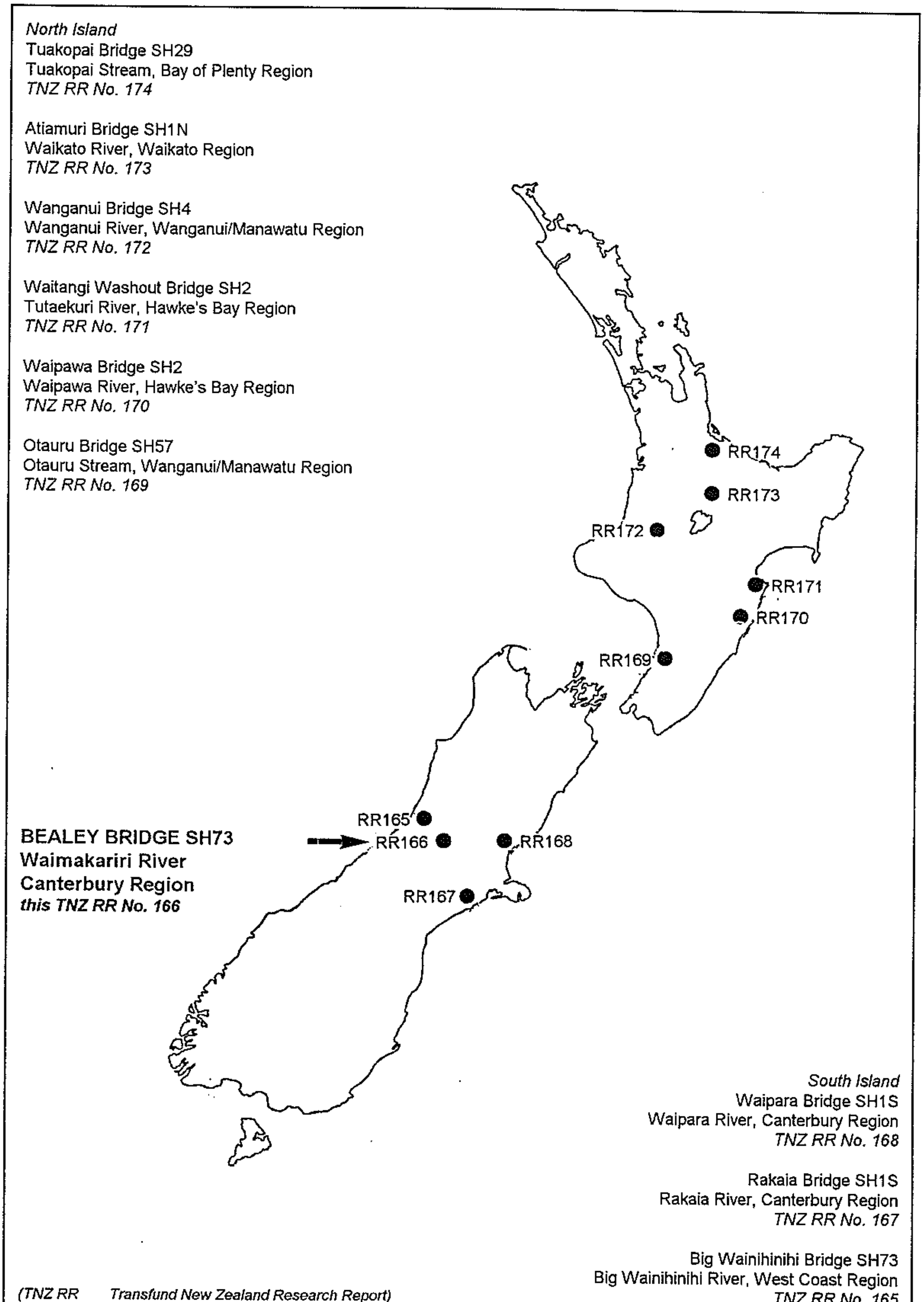
Further investigation may also be appropriate to determine why there is a significant difference in the behaviour of the girders in Span 2.

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 10 bridges on New Zealand highways, in order to develop and evaluate the methodology. The Bealey Bridge over the Waimakariri River, on State Highway 73 east of Arthur's Pass, Canterbury Region, South Island, was built in 1935. It was selected as one of these ten because it is an aging single-lane reinforced-concrete bridge with a concrete deck, and it has a low conventional strength rating (65%). Its Fitness for Purpose Evaluation indicates that the bridge is safely carrying the heavy vehicle traffic currently using the route.

Figure 1.1 Location of Bealey Bridge, Waimakariri River, 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, also called its "Fitness for Purpose". This method involves monitoring the response of a bridge to its normal environment, in particular to the traffic loads it has to withstand. Subsequently this data is processed and used to evaluate the bridge's Fitness for Purpose.

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

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

This 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 which, in this case, is on the main concrete girders.

This report is part of Stage 2 of the project, and presents results for the Bealey Bridge over the Waimakariri River, on State Highway 73 (SH73) east of Arthur's Pass, Canterbury Region, South Island, New Zealand (Figure 1.1). The reasons for choosing this bridge in the representative sample were:

- It is an aging (built in 1935) single-lane, reinforced-concrete structure;
- It has a low conventional strength rating, on a route with low volumes of heavy vehicle traffic.
- It also has some flexural cracking in the main girders.

The objective of this investigation was to evaluate the Fitness for Purpose of the superstructure of the Bealey (Waimakariri) Bridge using the conventional evaluation technique and the proposed Health Monitoring techniques, and to compare the results of both techniques. The fitness of the bridge to carry the ambient heavy vehicle traffic loading 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:

- 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 superstructure to the ambient heavy vehicle traffic passing over the bridge for at least 24 hours (Health Monitoring).
- Recording the response of the superstructure 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.
- Subsequently, the Health Monitoring evaluation was compared with the conventional rating.

This report outlines the results of the analysis, the details of the Health Monitoring programme, and the Fitness for Purpose Evaluation of the superstructure based on the health monitoring results.

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

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

The bridge 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.

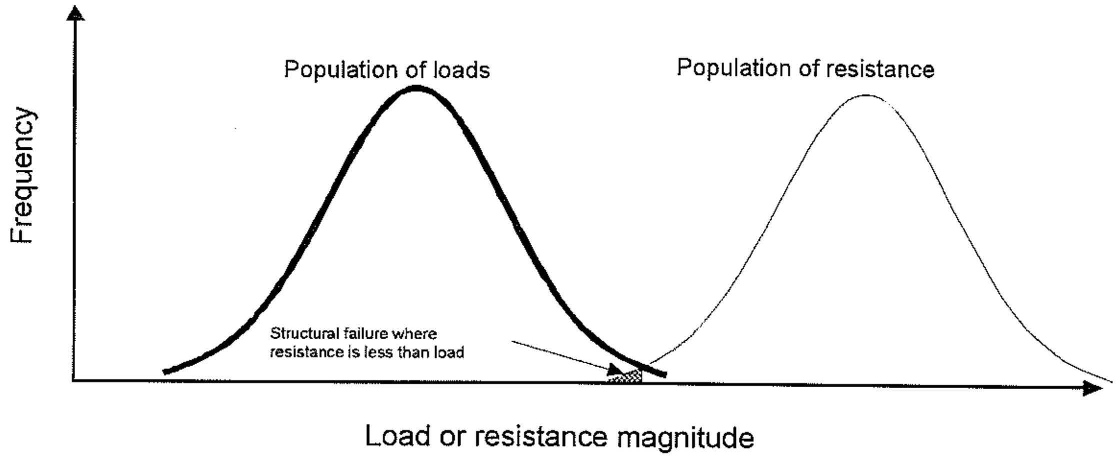


Figure 2.1 Statistical representation of structural failure.

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 often little 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** using 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 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. appropriate for conforming vehicles).*

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

2.3 Member Capacity & Evaluation using TNZ Bridge Manual Criteria

The Bridge Manual deals with main members and decks of a bridge separately. The evaluation approach described in Section 6 of the Manual is summarised here.

2.3.1 Main Members

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

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

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 that bridge.

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

2.3.1.2 *Posting Evaluations*

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

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

2.3.2 Decks

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

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

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

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

2.4 The Health Monitoring Approach

2.4.1 Theory of this Approach

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

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

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

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

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

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

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

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

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

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

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

where:

FPE = Fitness for Purpose Evaluation
$\gamma_o R_o$ = Ultimate Traffic Live Load Capacity
$UTL \ Effect$ = Ultimate Traffic Load Effect derived from the 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 Bealey Bridge over the Waimakariri River during the Health Monitoring programme, and the results are given in section 4.4 of this report.

3. Bridge Description & Assessment

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

3.1 Bridge Description

The Bealey Bridge is located on SH73 where it crosses the Waimakariri River between Bealey and Arthur's Pass, in Canterbury Region. The structure consists of twenty simply supported spans, each with two reinforced-concrete girders, supporting a single-lane, reinforced-concrete deck. The concrete deck has integral kerbs. The typical span length is 13.4 m with the two approach spans being 13.4 m and 13.2 m in length. Construction of the 268 m-long bridge was completed in 1935. The bridge is illustrated in Figure 3.1, and the superstructure of the bridge is illustrated in Figure 4.2.

Figure 3.1 Bealey Bridge over the Waimakariri River, Canterbury, South Island.

A passing lane including a wider superstructure with three girders has been included approximately half way along the bridge. The sharp horizontal curve at the eastern Bealey (or Christchurch) end of the bridge restricts vehicle speeds on the bridge at that end.

The speed limit over the bridge is 60 km/h, although the sharp turn and limited width of the bridge restricts the traffic to speeds usually less than 40 km/h, particularly for heavy vehicles.

One of the piers on the Bealey end of the bridge subsided during the 1990s (Figure 3.2) and the two defective spans were temporarily bridged with Bailey bridge components. The load evaluation and monitoring did not include this span of the bridge, which was replaced in 1999.

Figure 3.2 Failed spans at the Bealey (eastern) end of the bridge.

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

- Bridge (superstructure) Class 65%
- Deck Capacity Factor (DCF) 0.89

These ratings are based on the evaluation methods set out in Section 6 of the Bridge Manual, which are 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 rating and posting loads (see section 2.1 of this report), as specified in the Bridge Manual. Results from an analysis using the "known vehicle" (section 2.4.2) used in the Health Monitoring programme are also included. Details of this known vehicle are given in section 4.2 of this report.

A typical span of the bridge superstructure was investigated using a "grillage analysis"[1]. The grillage analysis assumed that the girders are simply supported. The dimensions of the structure used in the analysis were taken from the "as constructed" plans, which were confirmed by on-site measurements.

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

- Concrete f'_c = 17 MPa; E = 25 200 MPa
- Steel Reinforcement f_y = 250 MPa, E = 200 000 MPa

3.2.1 Girder Bending

The maximum bending moment in the girders resulting from the dead load is 440 kNm/girder. In this case the bending due to the dead load is the same for each girder. A summary of the maximum bending moments resulting from the various loads applied to the grillage model is presented in Table 3.1. The results in the table are not factored, and they represent the maximum bending moment in a single girder with the vehicle at the greatest allowable eccentricity. The 0.85 HO vehicle loading caused the maximum response.

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

Load	Bending Moment (kNm)
Dead Load	440
Known Vehicle	250
0.85 HN Vehicle	353
0.85 HO Overload Vehicle	523

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 1120 kNm.

3.2.2 Girder Shear

The shear force in each girder was found using the grillage analysis, and the results are presented in Table 3.2. The shear capacity (ϕV_n) of the main girders, found in accordance with Section 9 of the Concrete Structures Standard (NZS 3101: Part1 1995), is 440 kN.

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

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

Load	Shear Force (kN)
Dead Load	132
Known Vehicle	82
0.85 HN Vehicle	116
0.85 HO Overload Vehicle	168

3.2.3 Deck Capacity

Generally the deck on this bridge is not subjected to wheel loads because:

- The bridge is a narrow one-lane bridge.
- The wheel paths of heavy vehicles correspond to the girder positions and therefore the wheel loads are transferred directly into the girders.
- The deck of the span that was monitored did not have a passing lane section and would not have been subjected to wheel loads. Therefore the deck was not monitored.
- The span with the passing lane was not monitored, but its deck would be subject to wheel loads in the usual fashion.

The deck is not considered to be a critical component of the superstructure, except in the passing lanes.

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 load evaluation of the structure are presented in Table 3.3. The rating and posting evaluations have been assessed for the bending and shear in the girders only. The table also presents a comparison of the load rating calculated by Infratech Systems & Services (Infratech), and the load rating in the current (1999) TNZ Structural Inventory. A value of 1.3 was used for the impact factor in calculating the load ratings, and a value of 1.25 was used for the dead load factor.

The overall rating of the girders is taken as the minimum rating of all the components. For this bridge, the rating (56%) is the minimum of the ratings based on shear and bending, and the critical failure mode is midspan bending of the girders. This result does not compare particularly well with the rating of 65% which is documented in the TNZ Structural Inventory. The differences may be associated with the assumptions made regarding material properties.

Because the posting evaluation is less than 100 % (i.e. 66%), the expected normal practice would be to post this bridge with a load limit. It is understood that this bridge is not currently posted.

Table 3.3 Summary of theoretical load evaluations for the superstructure.

Mode of Failure	ϕUltimate 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	1120kNm	523kNm	353kNm	440kNm	56%	66%	65%
Girder Shear	440kN	168kN	116kN	132kN	79%	90%	–

3.4 Summary

The Bealey Bridge, in Canterbury, 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 bending moment in the girders was found to govern the strength and therefore determines the rating of the superstructure.

Based on the results from this analysis, the Health Monitoring programme concentrated on evaluating the Fitness for Purpose for the girders based on midspan bending. The bending in the deck was not measured because the deck would not generally be subject to wheel loads.

4. Health Monitoring Programme

The programme of Health Monitoring on the Bealey 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 for the Bealey Bridge are presented in this section of the report.

4.1 Instrumentation

The instrumentation installed on the bridge included eight Demountable Strain Gauge transducers. The locations of these transducers are illustrated in Figure 4.1.

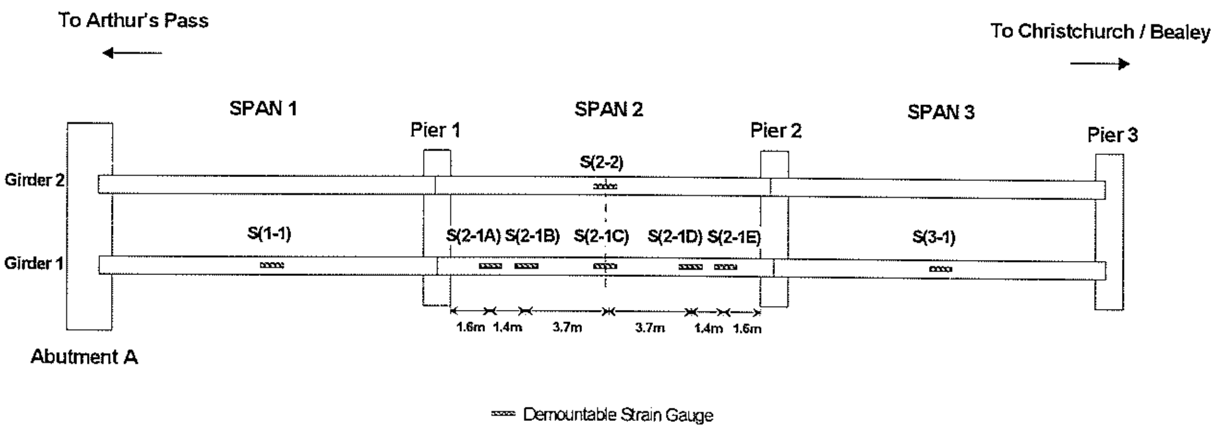


Figure 4.1 Instrumentation plan for the Bealey Bridge.
(S - strain transducer)

The terminology for the instrumentation is presented in Table 4.1. Figure 4.2 shows the instrumentation installed on Girder 1, Span 2. This girder was instrumented with five Demountable Strain Gauge transducers along its length. This strategy was used to record the variation in strain along the girder length.

The response of the deck slab was not instrumented. As mentioned previously, the wheel of a vehicle cannot physically cause major bending in the deck in this span.

Table 4.1 Summary of transducer identification names and positions.

Transducer ID	Position Name
S(1-1)	Span 1 Girder 1
S(2-1A)	Span 2 Girder 1a
S(2-1B)	Span 2 Girder 1b
S(2-1C)	Span 2 Girder 1c
S(2-1D)	Span 2 Girder 1d
S(2-1E)	Span 2 Girder 1e
S(2-2)	Span 2 Girder 2
S(3-1)	Span 3 Girder 1

Each demountable strain gauge was positioned over a crack in the soffit of the girder to which it was applied. Figure 4.3 illustrates the position of a demountable strain gauge over a typical crack in the girder. The site inspection of the structure revealed a large amount of flexural cracking in the girders of each span. This indicates that the service loads have exceeded the cracking strain in the concrete.

Figure 4.2 Instrumentation on Girder 1, Span 2.

Figure 4.3 Demountable strain gauge transducer placed over a crack in the girder.

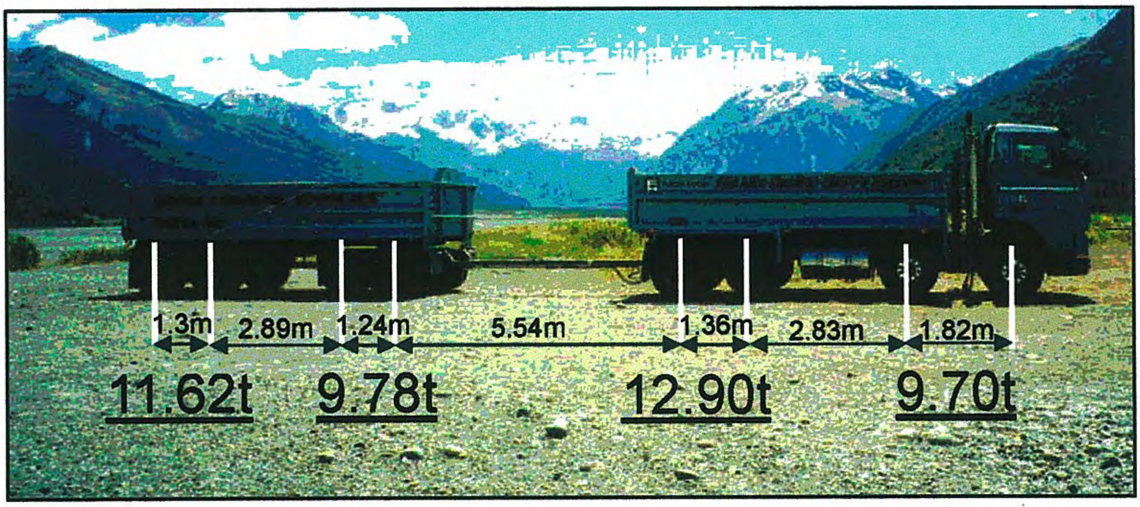
The demountable strain gauges (gauge length 230 mm) used on the girders measured strain at a point 20 mm below the soffit of the girders The results have been corrected to represent the strain in the soffit of the girders. The sign conventions used throughout this report include positive values for tension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

4.2 Procedure

The Health Monitoring of the structure began on Thursday 26 November, and continued until Sunday 29 November, 1998, giving a total monitoring period of approximately 72 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 125 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 Friday 27 November, 1998. The vehicle used for the testing was supplied by Fulton Hogan Ltd, and is shown in Figure 4.4, which also shows the axle weights and configuration of this 8-axle truck and trailer combination. The vehicle was not loaded to the legal limit for this configuration, and its gross mass was 44 tonnes.

Figure 4.4 The known vehicle used for behavioural testing.



The testing with the known vehicle was conducted by recording the response of the bridge to the vehicle as it passed over the bridge at different speeds. The tests were conducted with the vehicle travelling in both directions (east, then west) from a crawl (10 km/h), 20 km/h, to 40 km/h.

The lateral position of the known vehicle was in the normal lane as shown in Figure 4.5. Testing was completed by slowing the traffic in each direction or in some cases stopping it for a few minutes at a time. This ensured minimal traffic interruptions and also allowed the continuous monitoring of ambient heavy vehicles between the test runs with the known vehicle.

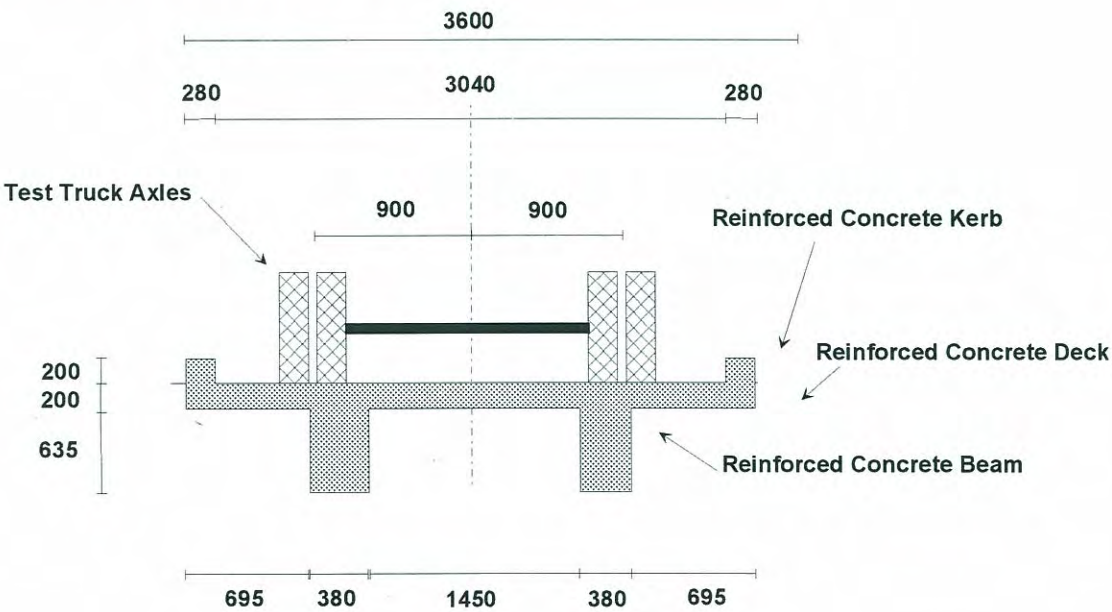


Figure 4.5 Lateral position of known vehicle during testing.

Figure 4.6 shows the known vehicle on the bridge and the small amount of lateral clearance (approximately 420 mm) available for the vehicle on each side. This restricts the path of the vehicle, which in turn limits the variation in the lateral distribution of load into each girder.

Figure 4.6 Known vehicle on the bridge during testing.

4.3 Short-Term Health Monitoring Results

4.3.1 Girder Response

A typical strain response versus time was graphed (as waveforms) for the midspan bending strains (Span 2, Girder 1) recorded during the Health Monitoring for the passage of a heavy vehicle. The response is presented in Figure 4.7. The waveforms show some continuity between the spans, evident by the small compressive strains before and after the vehicle passes over the instrumented span.

The scatter diagram for the midspan transducers (Figure 4.8) represents the maximum strain recorded during the passage of each heavy vehicle for the entire Health Monitoring period. These plots give an indication of the characteristics of the characteristics of the heavy vehicles travelling over the bridge, including distribution of mass and the number of heavy vehicles travelling this route. The scatter diagram also illustrates the difference in the volume of traffic on the weekdays (26-27 Nov.) and the weekends (28-29 Nov.).

Figure 4.9 Strain response versus time for transducers installed on Spans 1 and 3, Girder 1, and Span 2, Girder 2, for event recorded at 18.10, 26 Nov 1998.

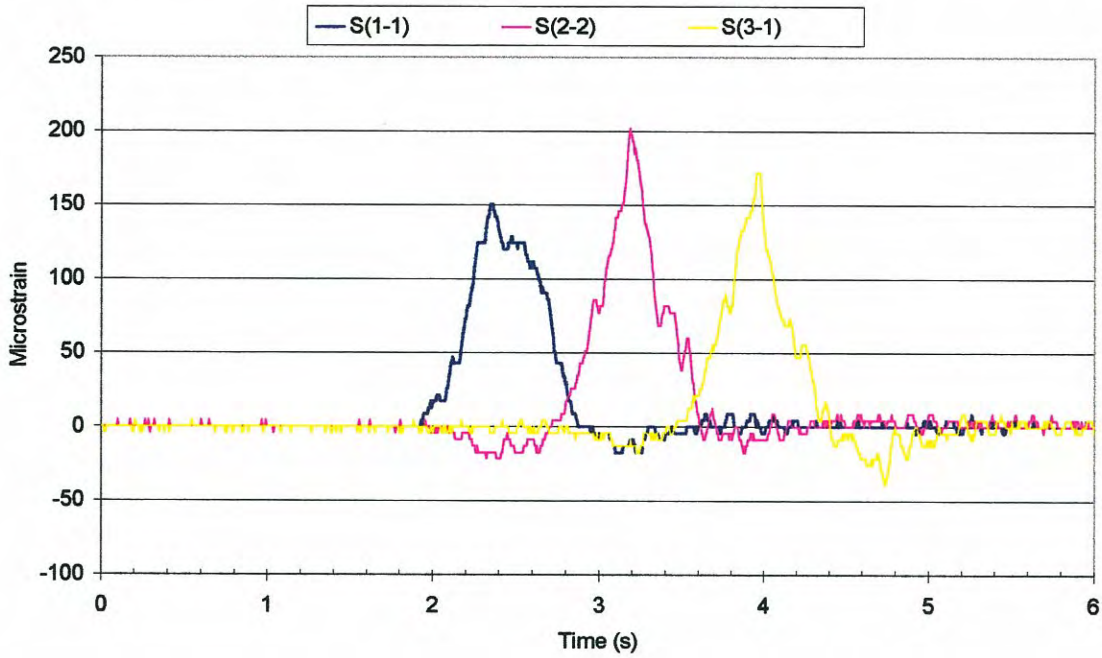
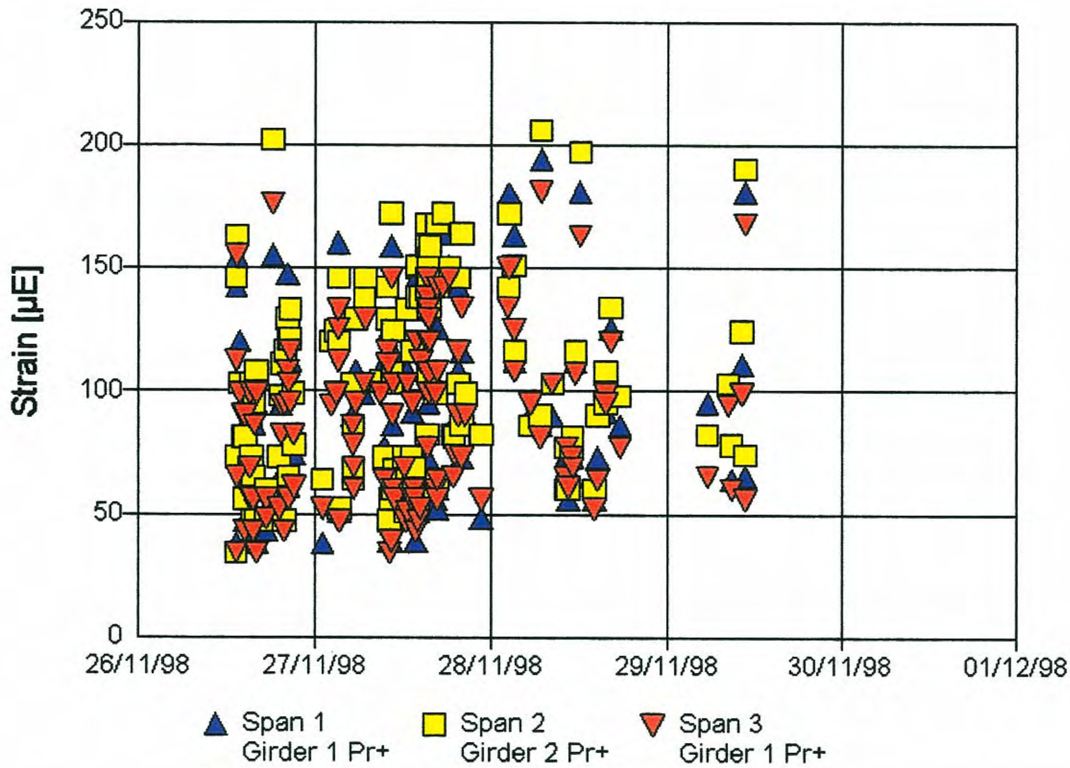


Figure 4.10 Scatter diagram for transducers installed on Spans 1 and 3, Girder 1, and Span 2, Girder 2.



The scatter diagram presented in Figure 4.8 indicates that the greatest response was from transducer S(2-1C) at the midspan of the girder, which is to be expected. The diagram also presents the response of the other four transducers installed on the girder. A comparison of transducers S(2-1B) and S(2-1D) shows that, in each case, the magnitude of these transducers is different, with transducer S(2-1B) the larger for every event. This is discussed further in section 4.3.2 of this report.

The waveforms for the remaining three transducers in Spans 1, 2 and 3 are illustrated in Figure 4.9. The plot shows the response of the structure as the vehicle travels from Span 1 towards Span 3 (east towards Bealey). The continuity is evident between all three instrumented girders. A scatter diagram for these transducers is presented in Figure 4.10.

4.3.2 Strain Distribution

The distribution of strain along Span 2, Girder 1, is presented in Figure 4.11. It shows that the largest response is occurring at the midspan transducer (S(2-1C)). Another significant aspect of the load distribution along the girder is the difference in the distribution recorded by the transducers located at equal distances from the midspan of the girder. Generally, locations at an equal distance from the midspan would be expected to experience the same response. In this case, transducer S(2-1B) is consistently larger than S(2-1D) and similarly transducer S(2-1E) is larger than S(2-1A). These differences could be related to:

- Variation in flexural cracking along the girder.
- The degree of restraint at the supports is different at each end of the girder.
- Heavy vehicles travelling in one direction only

A comparison of the lateral strain distribution between Girders 1 and 2 in Span 2 for the 20 maximum strain events recorded during the health monitoring (based on maximum response) is presented in Figure 4.12. The distribution shows 60% of the total strain is in Girder 2 and the remaining 40% of the strain is in Girder 1. The diagram verifies that the direction of travel and lateral positioning of the vehicle does not affect this distribution to a large extent. Since similar loads produce significantly larger effects in Girder 2 compared with Girder 1, Girder 2 appears to have been subject to greater deterioration (and is therefore in worse condition) than Girder 1.

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, for transducer S(2-1C), is presented in Figure 4.13.

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 installed on Span 2, Girder 1, are presented in Figure 4.14.

Figure 4.11 Strain distribution along Span 2, Girder 1 of Bealey Bridge.

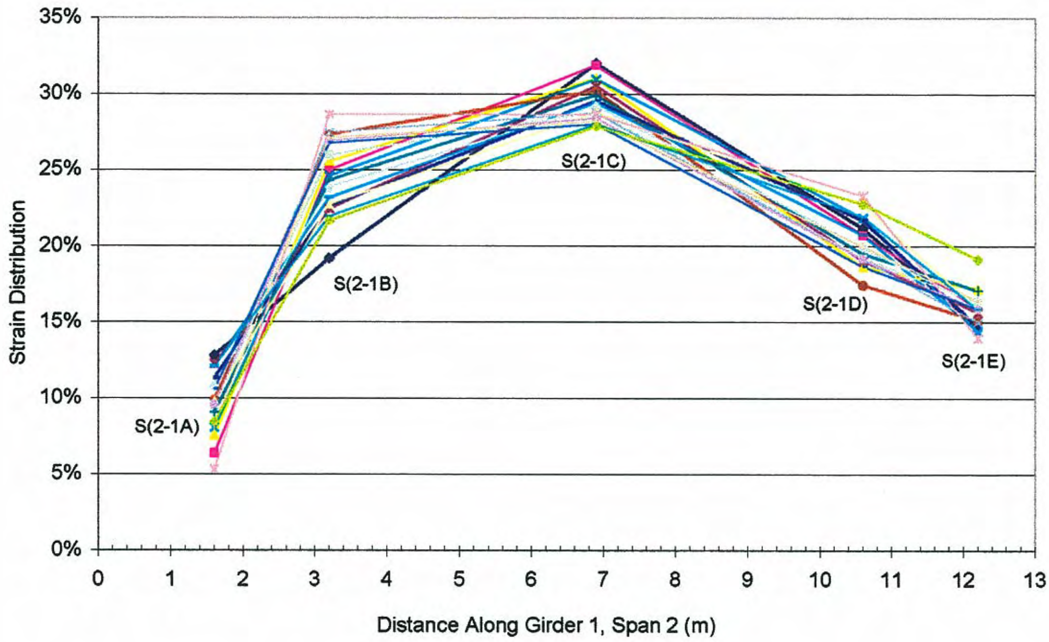


Figure 4.12 Distribution of strain between Girders 1 and 2 of Span 2.

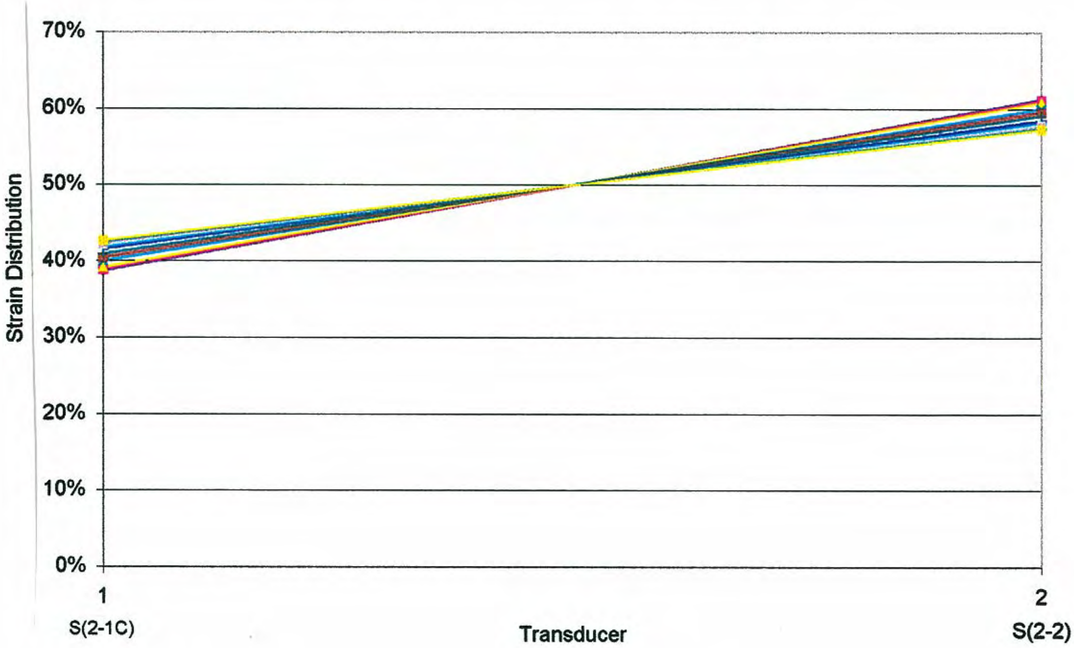
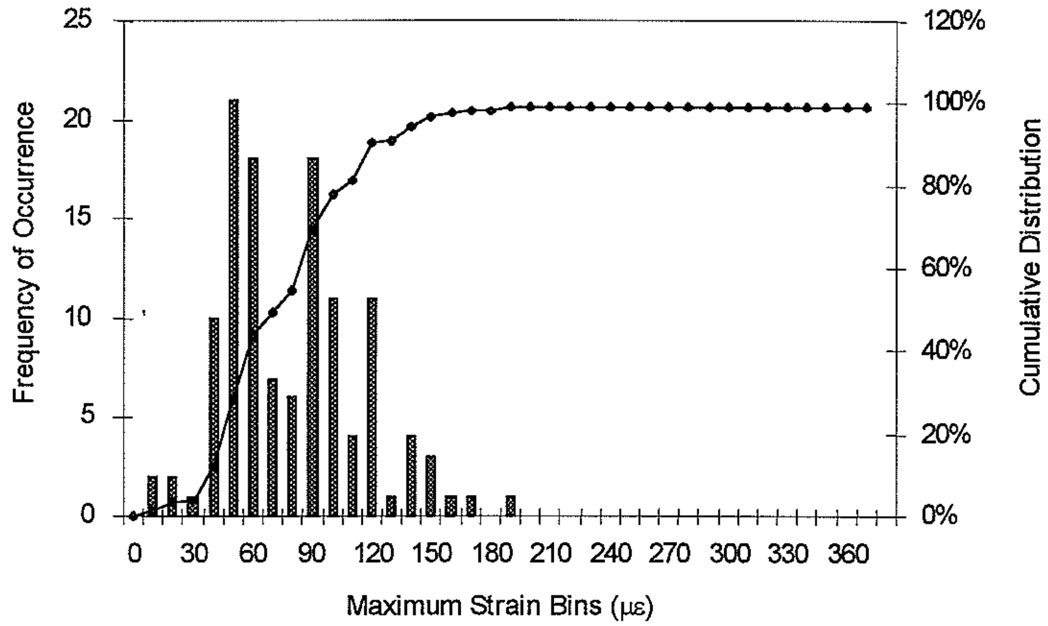


Figure 4.13 Histogram and cumulative distribution function for the transducer installed on Span 2, Girder 1C.



In Figure 4.14 the vertical scale represents the number of standard deviations that each point is away from the mean. The horizontal scale is the maximum strain recorded for each event. The point at which a data plot crosses the horizontal axis represents the average (mean) strain. A straight line represents a normally distributed sample of data.

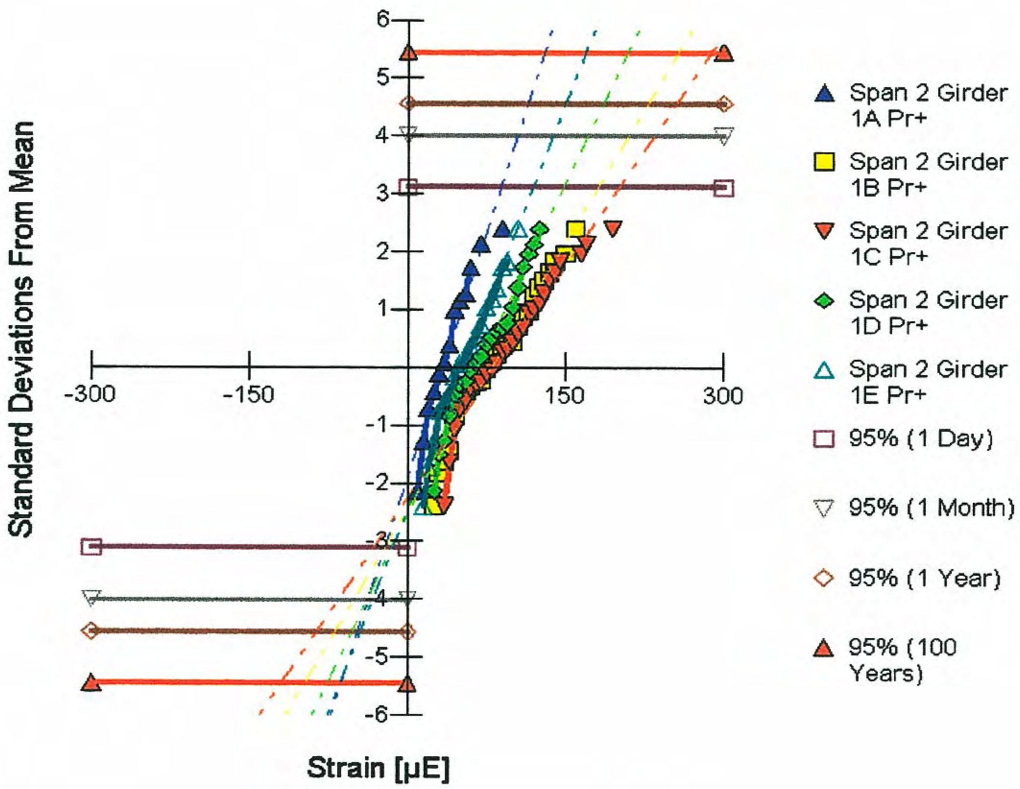
Horizontal lines representing the expected position of the 95% confidence limit for the data for 1 day, 1 month, 1 year, and 100 years have been plotted. Extrapolating the recorded data allows estimates of strain for these longer return intervals. The strain extrapolated for the 95% confidence limit for 100 years represents the ultimate traffic load effect for the Fitness for Purpose Evaluation as outlined in section 2.4 of this report.

The inverse normal shows that the strain extrapolated for the 95% confidence limit for 100 years (ultimate traffic load effect) is the greatest for the midspan transducer S(2-1C). The extrapolated value is approximately 300 με.

The inverse normal plots for the remaining transducers on Spans 1, 2 and 3 are presented in Figure 4.15. The extrapolated results show very similar values with transducer S(2-2) showing that the greatest extrapolated event was equal to approximately 340 με.

The maximum results along with the extrapolated results for all transducers are presented in Table 4.1. The table shows the extrapolated 95% confidence limit for 100 years for each transducer installed on the bridge. The extrapolated events range from 300 με to 350 με for the midspan transducers.

Figure 4.14 Inverse normal plot for transducers installed on Span 2, Girder 1.



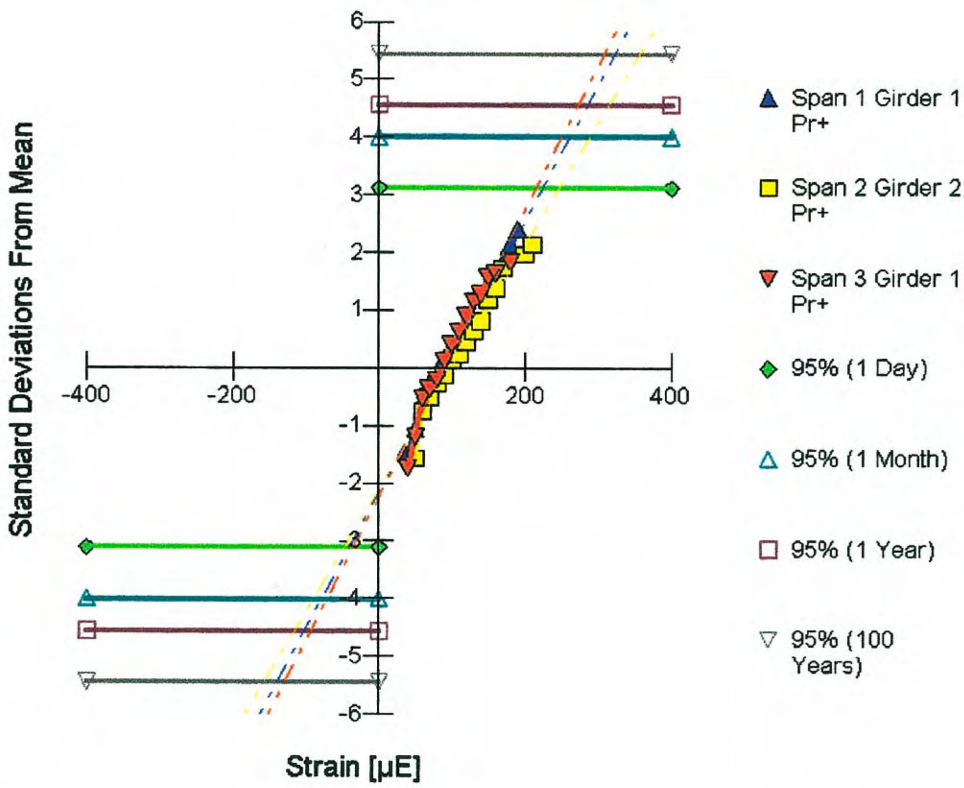


Figure 4.15 Inverse normal plot for transducers on Spans 1 and 3, Girder 1, and Span 2, Girder 2.

Table 4.2 Extrapolated data obtained from inverse normal plot.

Transducer	Maximum Recorded Value (Health Monitoring)	Extrapolated Value (95% Confidence limit) for 1 year	Extrapolated Value (95% Confidence limit) for 100 years
	Strain ($\mu\varepsilon$)		
S(1-1)	195	290	320
S(2-1A)	90	120	130
S(2-1B)	160	230	260
S(2-1C)	195	260	300
S(2-1D)	125	190	210
S(2-1E)	105	150	170
S(2-2)	205	305	350
S(3-1)	180	270	310

4.4 Known Vehicle Testing

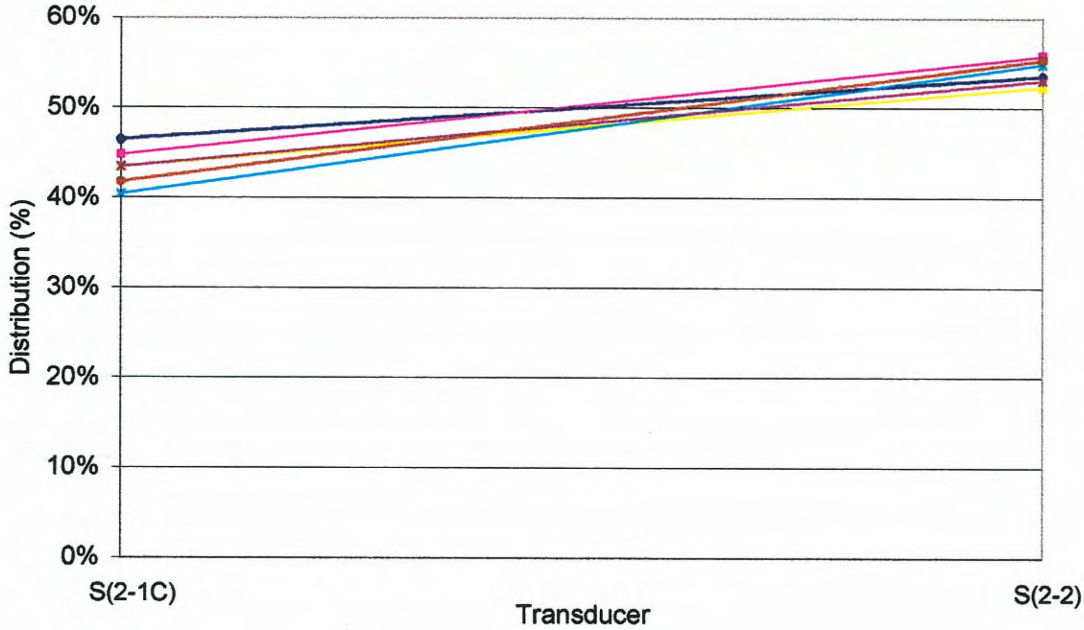
The known vehicle testing was performed at vehicle speeds ranging between a crawl and 40 km/h. The maximum strains recorded for each transducer during the testing with the known vehicle are presented in Table 4.3.

The distribution of load into each girder is presented in Figure 4.16. The distribution presented is relatively consistent with the data collected from health monitoring of the ambient heavy vehicle traffic.

Table 4.3 Maximum recorded strains for known vehicle testing.

Transducer	Maximum Strain
S(1-1)	160$\mu\varepsilon$
S(2-1A)	60$\mu\varepsilon$
S(2-1B)	135$\mu\varepsilon$
S(2-1C)	140$\mu\varepsilon$
S(2-1D)	110$\mu\varepsilon$
S(2-1E)	90$\mu\varepsilon$
S92-2)	170$\mu\varepsilon$
S(3-1)	145$\mu\varepsilon$

Figure 4.16 Distribution of strains for known vehicle testing.



The dynamic response of the main girders was small because of the relatively slow speeds of the vehicle. The very narrow width of the structure ensures that vehicle speeds are restricted to less than 40 km/h. The driver of the known vehicle commented that it was very difficult to achieve speeds in excess of 40 km/h because of the narrow bridge width.

The free vibration response is small, being approximately 3% of the maximum response. Because this response is small the level of damping could not be determined. The natural frequency of the superstructure is around 7 Hz. Higher resolution settings on the monitor may have given better signals to allow this frequency to be determined more accurately.

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 (AUSTROADS 1992) was calculated using the following equation:

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

The response of the crawl test was used for the static result in the calculation of dynamic increment. In the case of the Bealey Bridge, the maximum dynamic increment was calculated for Span 1, Girder 1. All other girders in Spans 2 and 3 displayed very small or negative dynamic increments. A negative dynamic increment indicates that the worst-case effects occur when the vehicle is travelling at lower speeds. The dynamic increment for Span 1, Girder 1, was found to be only 7% and was calculated for the known vehicle travelling east towards Bealey.

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 is to be expected as the known vehicle was not loaded to the legal limits. On average the maximum recorded value for the health monitoring was 25% higher than that recorded for the known vehicle.

Table 4.4 Summary of health monitoring data.

Transducer	Maximum Recorded Value (Known Vehicle)	Maximum Recorded Value (Health Monitoring)	Extrapolated Value (95% Confidence limit) for 1 year	Extrapolated Value (95% Confidence limit) for 100 years
	Strain ($\mu\varepsilon$)			
S(1-1)	159	195	290	320
S(2-1A)	60	90	120	130
S(2-1B)	133	160	230	260
S(2-1C)	138	195	260	300
S(2-1D)	111	125	190	210
S(2-1E)	90	105	150	170
S(2-2)	168	205	310	350
S(3-1)	146	180	270	310

5. Fitness for Purpose Evaluation

5.1 Main Girders

The structural analysis 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 available to resist the ultimate traffic live load was 568 kNm ($\phi M_u - 1.25DL$).

The extrapolated health monitoring response for the midspan bending strain recorded at the soffit of the girders is presented in section 4.3 of this report. The result for the 95% in 100 years events is 350 $\mu\varepsilon$ (Table 4.4). For the Bealey Bridge, each demountable strain transducer was installed directly over a crack in the soffit of the girder. For this bridge therefore, the measurement recorded by the demountable strain transducer 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 tensile bending strain in the reinforcement in the girders.

The crack width model used in this research 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 the reinforcement
h - cover to the bottom level of reinforcement
A - concrete tension area surrounding reinforcing bars

Parameters are:
D - overall depth of the concrete section
d - depth to the centroid of the reinforcement
k - neutral axis parameter (also k_u)

The maximum extrapolated event (95% in 100 years) is 350 $\mu\varepsilon$ over a gauge length of 230 mm. This corresponds to a crack width movement (w_{max}) of 0.079 mm. Substituting this in Equation 8, along with the appropriate values of h and A for the concrete girder, gives a stress in the reinforcement of 87 MPa. This corresponds to a strain in the steel of 430 $\mu\varepsilon$ and represents an increase of 26% over the recorded soffit strain. Consequently the recorded soffit strains in this report must be increased by 26% to represent the actual tensile bending strain in the reinforcing steel.

Figure 5.1 illustrates a theoretical moment versus strain curve for a typical girder of the Bealey Bridge. The graph summarises the method used by Infratech to obtain a relationship between bending moment and strain for determining the Fitness for Purpose Evaluation for this bridge.

[2] ACI Australian Concrete Institute

This method is applied because the relationship between bending moment and strain is not linear, as the stiffness changes after the concrete beam cracks.

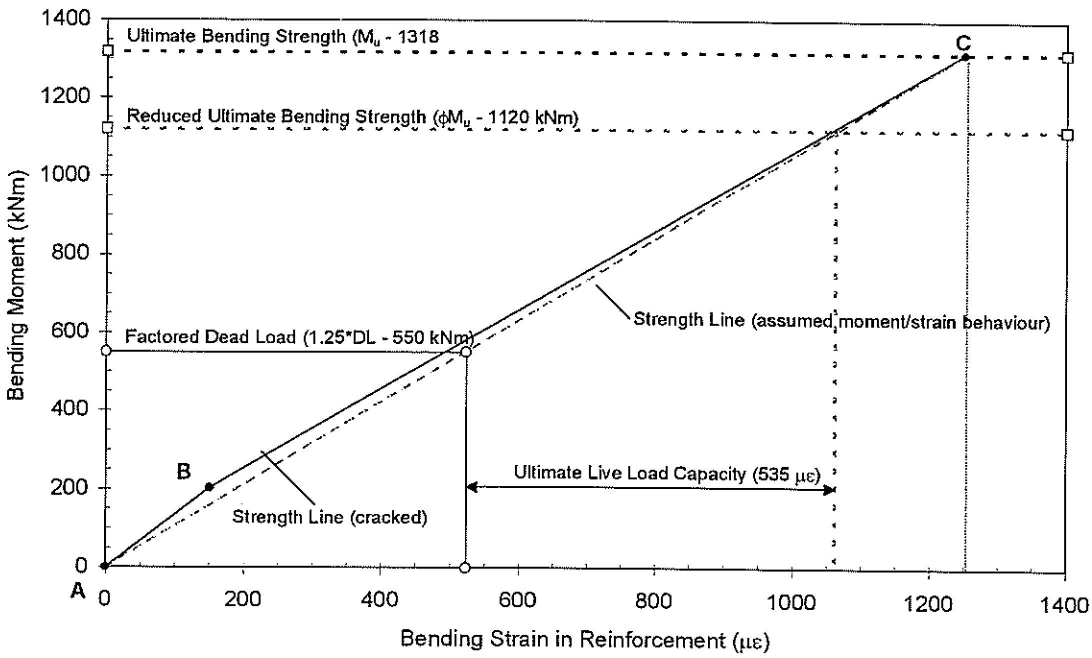


Figure 5.1 Theoretical moment versus strain relationship for the Bealey Bridge.

The Line AB on Figure 5.1 represents the linear elastic behaviour of the concrete girder section before cracking occurs. Point B represents the point at which the concrete cracks. At this point the concrete begins to follow Line BC which represents the behaviour of the concrete girder section in the cracked state.

A conservative assumption of the behaviour for the concrete beam action is characterised by (dashed) Line AC. Because the girder is already cracked under service loads, the actual relationship between moment and strain for the girder is expected to be similar to the Line AC.

Figure 5.1 also presents the reduced capacity (ϕM = 1120 kNm) of a typical girder for this bridge converted to an equivalent strain (1060 $\mu\varepsilon$). This calculation is based on the theoretical moment versus strain relationship in the figure. The factored dead load moment (550 kNm) was converted to an equivalent strain equal to 525 $\mu\varepsilon$. This gives an ultimate live load capacity equal to 1060–25 = 535 $\mu\varepsilon$.

Table 5.1 summarises the calculation of the Fitness for Purpose Evaluation based on the health monitoring data and the information presented in Figure 5.1. The method for the calculation of this evaluation is 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 125%. This evaluation compares poorly with the theoretical rating evaluation calculated for the 0.85 HO rating (56%, Table 3.3) and for the 0.85 HN posting (66%) evaluations. The comparison with the 0.85 HN posting evaluation is the most appropriate as this evaluation is related to ambient heavy vehicle traffic.

Table 5.1 Summary of Fitness for Purpose Evaluation.

Item	Result
Strength (ϕM_n)	1120 kNm
Dead Load (*1.25)	550 kNm
Ultimate Live Load Capacity Moment ($\gamma_o R_o$)	570 kNm
Ultimate Live Load Capacity – Equivalent Strain ($\gamma_o R_o$)	535 $\mu\varepsilon$
Maximum Recorded Soffit Strain (Ambient Traffic)	205 $\mu\varepsilon$ (Girder 2, Span 2)
Ultimate Traffic Load Effect (95% in 100 years) (Soffit Strain)	350 $\mu\varepsilon$
Ultimate Traffic Load Effect (95% in 100 years – strain in steel reinforcement based on crack-width theory)	430 $\mu\varepsilon$
Fitness For Purpose Evaluation	125%

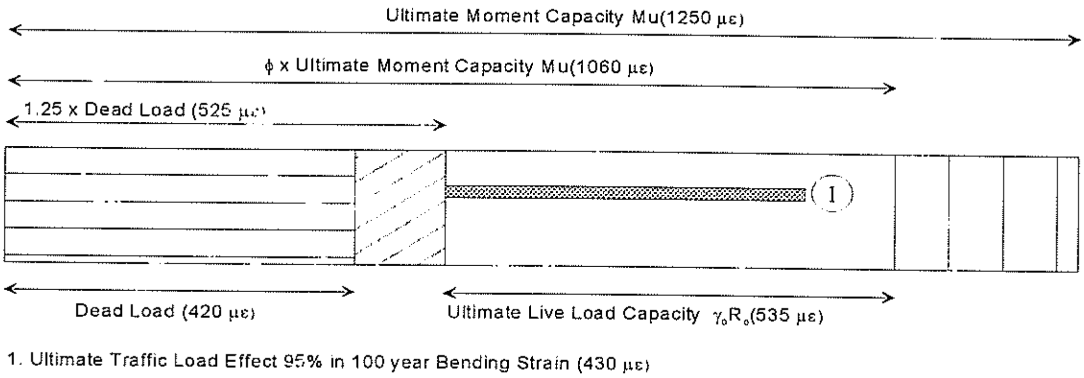


Figure 5.2 Summary of Fitness for Purpose Evaluation based on limit-state design principles.

A summary of the Fitness for Purpose Evaluation is presented in the capacity diagram of Figure 5.2. All strains in this diagram are in terms of the tensile bending strain in the cracked reinforced-concrete section.

The overall rectangle represents the ultimate capacity in bending (1060 $\mu\varepsilon$). The horizontally hatched section represents the dead load (420 $\mu\varepsilon$), and the diagonally hatched section represents the factored component required for the 525 $\mu\varepsilon$ factored dead load. The white unshaded section represents the capacity available for the ultimate traffic live load effect (535 $\mu\varepsilon$), and the dark grey section (1) represents the ultimate traffic load effect in 100 years (430 $\mu\varepsilon$).

5.2 Summary

The Fitness for Purpose Evaluation for the Bealey Bridge, based on midspan bending of the main girders, is 125%. The Fitness for Purpose Evaluation compares poorly with the evaluations based on the 0.85 HN posting load of 66% and the 0.85 HO rating load of 56%. The comparison with the posting load is the most relevant. The reasons for the higher rating evaluation obtained from the health monitoring include:

- The ambient heavy vehicles typically induced lower bending moments than the 0.85 HN vehicle for this span. The known vehicle induced bending moments around 75% of the 0.85 HN loading, while the maximum recorded ambient traffic induced strains approximately 88% of the 0.85 HN loading. Many of the heavy vehicles recorded on this route were significantly lighter than the known vehicle.
- The response of the structure to the ambient heavy vehicles showed that some continuity existed between spans, thus reducing the live load effect which increases the Fitness for Purpose Evaluation result. This continuity may be related to bearing restraint or arching effects.
- The actual concrete strength is probably higher than that assumed in the analysis.
- The bridge has a low dynamic response despite the uneven longitudinal profile. This low dynamic response is most likely the result of the slow speed of the heavy vehicles.

As mentioned in section 3.3 of this report, under normal practice this bridge would be posted, based on the theoretical posting evaluation. However the Health Monitoring of the main girders indicated that the bridge is currently Fit for Purpose, and that it is safely carrying the heavy vehicle traffic currently using this route. It is possible that the opening of the Otira viaduct on SH 73, in November 1999, may have influenced traffic characteristics since this data was gathered (i.e. more heavily laden vehicles may now be using the route). This possibility should be considered when management options for this bridge are being assessed.

This investigation has determined a Fitness for Purpose Evaluation based on the data recorded during Health Monitoring of the Bealey Bridge. This Evaluation is based on several constraints and conditions that are present now but may change in the future, such as:

- The reliability of the bearing restraint continuity effects.
- The change in heavy vehicle traffic characteristics and the possibility of increased heavy vehicle loads and volumes since the completion of the Otira Viaduct, on SH73, in November 1999.

6. Conclusions

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

Theoretical Analysis
The theoretical analysis of the bridge found that midspan bending of the main girders was the critical issue associated with the performance of the bridge. Hence the Health Monitoring programme focused on assessing the performance of the bridge based on this component.

The concrete deck, which was not considered in this investigation, has a DCF of 0.89, according to the TNZ Structural Inventory. The narrow width of the structure, together with the position of the wheel loads relative to the girders, means that the deck is generally not subjected to wheel loads except in the spans with passing lanes. Thus, deck strain was not monitored.

The theoretical assessment of the superstructure of the bridge found that the 0.85 HO rating was 56% and the 0.85 HN posting was 66%. These values compare reasonably well with the value of 65% for the 0.85 HO rating listed in the TNZ Structural Inventory. According to normal practice this bridge would be posted based on this assessment.

Health Monitoring Results
The results found that:

- Some continuity exists between spans, indicating some bearing restraint effects.
- Typical heavy vehicles are inducing bending moments in the bridge that are up to 90% of the 0.85 HN posting load. However, a significant portion of the heavy vehicles using this route are lighter than these vehicles, and also the traffic volumes are low.
- Because of the narrow width of the structure, the lateral position of the heavy vehicles is restricted and thus distribution of load in to each girder is consistent.
- An impact factor of 1.07 was found to be appropriate for this bridge. This is much lower than the impact factor of 1.30 used to determine the load rating that is detailed in the Bridge Manual. The impact factor is low because the road alignment and width restrict the vehicle speed.

Fitness for Purpose Evaluation

- The Fitness for Purpose Evaluation for this bridge, based on the critical midspan bending of the main girders, was 125%. This evaluation indicates that the bridge is safely carrying the heavy vehicle traffic currently using this route.
- The higher Fitness for Purpose Evaluation, compared with the posting evaluation, is mainly related to the lightly laden heavy vehicle traffic on this route and to the low impact factor.

7. Recommendations

The theoretical assessment of this bridge suggests that it should be posted with a load limit, although Health Monitoring results suggest that the bridge is performing adequately and that posting is not necessary.

However, traffic characteristics may have changed significantly since this data was gathered (in particular as a result of the opening of the Otira viaduct), and thus the Fitness for Purpose Evaluation may require revision.

Consequently the recommendation is to review the traffic characteristics.

Further investigation may also be appropriate to determine why there is a significant difference in the behaviour of the girders in Span 2.

8. References

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