University of Southern Queensland
Faculty of Engineering and Surveying

Testing and analysis of fibre composite beams in a bridge structure

A dissertation submitted by

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Courses ENG411 and 4112 Research Project

towards the degree of

Bachelor of Engineering (Civil)

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Abstract

The aims of this project were to investigate the behaviour of fibre composite beams under various load types (including dynamic loading) in a composite bridge structure, to determine whether simplified methods can be used to analyse the composite structure accurately, and to investigate issues involved in increasing the span of the bridge to full scale.

The testing was undertaken by the Centre of Excellence in Engineered Fibre Composites (CEEFC), in conjunction with the Queensland Government Department of Main Roads (DMR) on an approximately half-scale fibre composite bridge constructed by Loklite Pty Ltd in conjunction with the CEEFC.

Analysis of field results from this testing, and finite element analysis (FEA) using the Strand7 FEA software package was undertaken. Comparisons of results from both types of analysis were undertaken, and conclusions made from these comparisons were used to resolve the aims of the project.

Recommendations for areas of future research were also given, as this project has been shown to lead to a number of areas where more in-depth analysis and complex finite element modelling could provide greater insight into the behaviour of fibre composite beams in composite bridge structures.

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Christopher Ross Green

Student Number: 0050025553

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Date

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1. INTRODUCTION

1.1. Background

The use of fibre composites in the construction and transport industries is relatively new compared to their use in fields such as the aerospace, military and marine industries. Consequently, there has not been a great deal of research in this area, particularly in replacement of existing members in timber and concrete bridges.

This project will attempt to demonstrate that the replacement of beams in timber and concrete bridges by fibre composite elements is viable, both structurally and economically. The project will also attempt to determine whether the field testing results can be scaled up to the full design size from the prototype bridge being tested.

1.1.1. DMR and CEEFC

The Queensland Department of Main Roads (DMR) has been at the forefront in providing funding for research in this area, as it may become of economic importance to develop fibre composites rapidly to replace deteriorating bridge elements. The Centre of Excellence in Engineered Fibre Composites research group (CEEFC) has been studying fibre composites, particularly transport industry specific designs, for the last 11 years. The CEEFC and the DMR have been working closely together for much of this time to develop fibre composite beams and decks for specific bridge sites in the Queensland road network. The proposed site for the full scale bridge will be the third in the Australian road network, and second in Queensland.

1.2. Design and Analysis

As part of the project requires development of a testing plan, various papers describing bridge field testing have been reviewed to provide appropriate instrumentation and placement for this specific project. The designs of the beams and deck units have not been studied for this project, as these members have been prefabricated prior to testing.

The use of transverse stiffeners (diaphragms) will be studied to determine whether the analysis of the composite structure can be simplified using a grillage type system. Analysis using a simple grillage system is much quicker and simpler than developing a complex three-dimensional model for use in finite element analysis, but may not take some aspects of deck or beam behaviour into account. Comparison of results from both types of analysis may show that the simplified grillage system may be more useful in predicting deflections, due to the closeness of results and the time saving factor.

Investigation into the need for diaphragms will also be undertaken. This theoretical analysis will compare deflections calculated using different sized diaphragms with those calculated with no diaphragms present. These comparisons should give a good indication of whether diaphragms are necessary in the construction of fibre composite bridges.

1.3. Project Aims

This project aims to:

1. investigate the behaviour of fibre composite beams under dynamic and static loading in a composite structure;
2. determine whether simplified methods can be used to analyse the composite structure accurately; and
3. investigate issues involved in increasing the span of the bridge to full scale.

To achieve these aims, a number of objectives had to be met.

1. *Research the background information on previous field testing and instrument placement of bridge structures.*
2. *Develop a testing plan including placement of instrumentation on the beams and deck of the bridge, and static and live loading of the bridge.*
3. *Collect data from field testing of the bridge, as appropriate.*
4. *Analyse field data for use by Department of Main Roads, and compare field data with analysis using appropriate finite element software package (Strand7), taking deck effects into account.*
5. *Determine the viability of using simplified analysis methods (grillage analysis) to predict deflections accurately.*
6. *Investigate the issues involved in increasing the span of the bridge to full scale.*
7. *Given time, conduct a cost-benefit analysis into the viability of replacing hardwood timber bridge beams with fibre-reinforced polymer beams.*

1.4. Structure of Dissertation

The investigation of fibre composite beams in bridge structures will involve reviews of related studies, field testing of a small-scale bridge, analysis of field test data, finite element modelling and analysis, and comparisons between the field data analysis and finite element analysis. This section outlines the structure of the dissertation.

Chapter 2 contains reviews of previous studies undertaken that relate to this project. Studies include research into the use of fibre composite materials to replace hardwood and/or concrete bridge elements, the instrumentation and placement of instruments in field testing, types of analysis that have been previously used, and directions for further study into the use of fibre composite materials in the transport and construction industries.

Chapter 3 explains the methodology used for the project. This includes the development of the testing plan, involving the choice of and positioning of instruments, the loading and run patterns to be used, the field data o be analysed, and the finite element model analysis.

Chapter 4 involves the analysis and discussion of the field testing data collected. This includes both dynamic and static loading conditions, mid-span girder deflections and strains, differential girder-deck and differential deck panel deflections, girder deflection near abutments, longitudinal and transverse deck strains, and girder shear strains.

Chapter 5 involves the development and analysis of a finite element model of the bridge, starting with a basic grillage model, then modifying the model, and adding deck panels to the model to find out how much influence the deck has on deflections. This chapter will also investigate the viability of using predicted deflections from the finite element analysis in predicting deflection of the full-scale bridge.

Chapter 6 will compare the analyses from Chapters 4 and 5, and the results of these comparisons will be discussed.

Chapter 7 will provide a summary of the project, conclusions arrived at from the undertaking of the project, and areas for future research will be highlighted. Recommendations will be made on the viability of using finite element modelling to predict deflections and influence the production of appropriate bridge elements for use in the construction of the full-scale bridge.

The appendices provide supporting material to the research. These include the project specification, the test bridge site and specifications, a risk assessment of the field testing, field testing worksheets and plots, and Strand7 FEA files.

1.5. Summary

Due to the number of deteriorating timber and concrete bridges in Australia, a cost effective method must be found to either rehabilitate or replace unsafe elements of these bridges. The increased use of fibre composite beams to replace timber and/or concrete beams in deteriorating bridges needs to be considered to make this a cost effective solution. This project investigates the properties of fibre composite beams in a bridge structure under dynamic and static loading to determine the viability of using these beams as replacements for timber and/or concrete beams.

2. LITERATURE REVIEW

2.1. Introduction

A comprehensive review of the available literature relating to the project was undertaken, with emphasis placed on specific areas as outlined below.

2.2. Materials

There are several main reasons for the use of fibre composite materials in the transport industry, particularly in bridge construction. As Holloway and Head (2001) suggest, the civil infrastructure market is controlled by existing materials with well known properties that can be easily manufactured. The challenge for manufacturers of fibre composite materials is to overcome the lack of performance data and cost concerns to show that production will be more environmentally sustainable than materials such as timber and steel, making fibre composites more economically viable for whole-of-life duration.

Ayers and Van Erp (2002) stated that the lack of accepted design standards for composite materials inhibits the usage of these materials into the mainstream construction industry. A search of both the Building Code of Australia (BCA 2007) and Standards Australia show that this is still the case, with the only design standard available being for the tensile properties of fibre reinforced polymers (FRPs) (AS 1145.4, AS 1145.5, 2001).

A case study was undertaken in 2002 by Cooperative Research Centre (CRC) for Construction Innovation to compare current (at the time) DMR practices and how FRP technology can be applied to these practices in the strengthening of reinforced concrete bridges. The target of this case study was rehabilitation of deteriorating headstocks, so while not aimed specifically at this project, the study shows that the DMR is prepared to look at new technology to construct new bridges and rehabilitate existing bridges in the road network.

The original FRP bridge was designed by USQ's Fibre Composites Design and Development team (FCDD, has since become the CEEFC) and installed at a quarry at Wellcamp, outside Toowoomba, in January 2002. This bridge was developed as an hybrid concrete-composite section (Van Erp et al. 2002), was being frequently monitored, and was still performing above expectation, with approximately 150 trucks crossing per day (Innovation Case Study No 5, CRC for Construction Innovation). In September 2002, the prototype design was awarded a "Highly Commended" by the Institute of Engineers, and in 2005 received a Nova Award nomination which recognises construction innovation.

Figure 2-1: Australia's first fibre composite bridge (Van Erp et al., 2002)

This bridge was a stepping stone to the current design, as was the replacement of 12m of existing concrete bridge deck by FRP deck on the Coutt's Crossing Bridge in northern NSW. This was the first use of FRP materials on a bridge in the Australian road network (Innovation Case Study No 5), and showed significant time and cost savings for installation and maintenance over traditional concrete deck replacement. The installation in June 2005 of a two-span (10m and 12m) FRP deck on traditional concrete substructure at Taromeo Creek, at Blackbutt, Queensland, to replace an existing timber bridge, is the largest FRP project to date (Fibre Composite Projects, Technical Note 54, March 2006).

Dunn et al. (2005) reported on the construction of the first steel-free bridge deck in the United States, in Tama County, Iowa. The use of FRPs in this bridge was predominantly to reduce the effects of corrosion of reinforcing steel by de-icing salts, which causes the surrounding concrete to deteriorate. The new deck will have increased durability, leading to lower maintenance costs, and should have lower whole-of-life costs than traditional concrete decking.

Figure 2-2: Elevation view of the Tama County Bridge (Dunn et al., 2005)

In April 2005, installation of fibre composite girders on the Heifer Creek No 5 Bridge, an existing timber bridge, was completed. These girders were a softwood-FRP hybrid, and had been comprehensively tested (Fibre Composite Projects).

2.3. Testing and Instrumentation

The natural frequency and damping ratio of the bridge is dependent on its length, as shown by Moses et al. (1992) and Samman et al. (2001). They are also dependent on the stiffness of the material (Young's modulus).

The positioning of strain gauges to measure longitudinal strain (tension and compression) will be placed similar to gauges in testing performed by Dunn et al. (2005), Fu and Harwood (1999), Konda (2003), and Potisuk and Higgins (2007).

However, Watkins et al. (2001) embedded fibre-optic sensors during production to measure temperature, flexure strain and shear strain. This technology is still new, and fairly expensive, so could not be justified for this project.

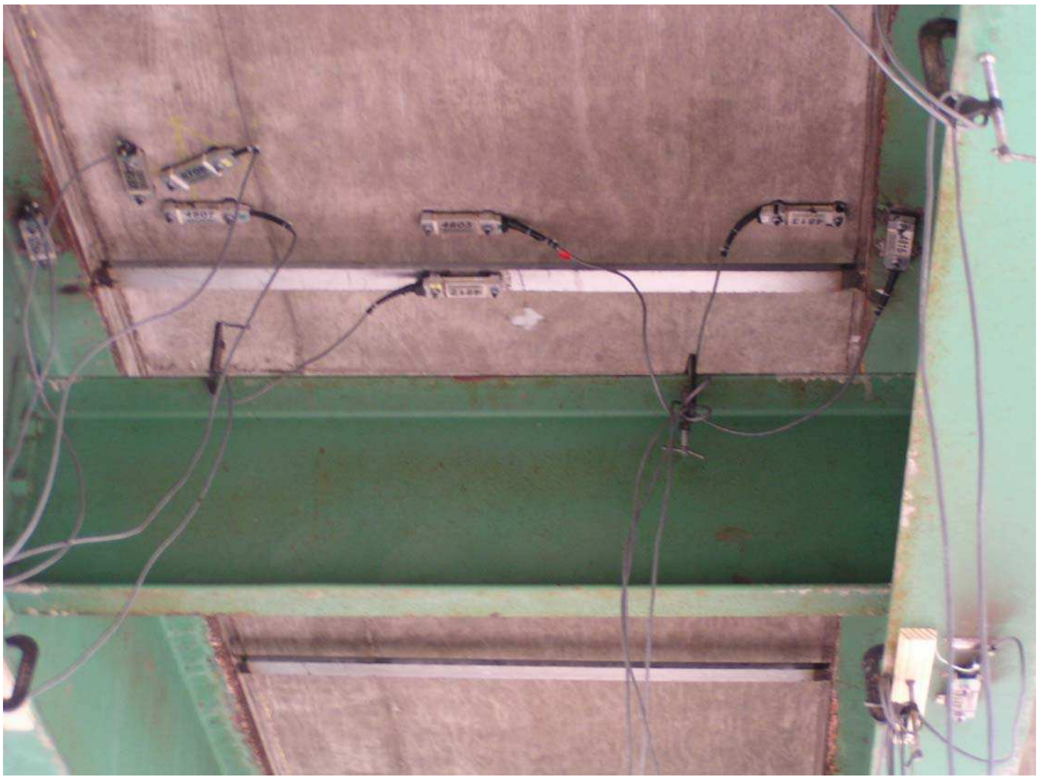

Figure 2-3: Placement of strain gauges (Dunn et al., 2005)

Positioning of string pot displacement gauges will be at mid-span, under beams and deck, similar to those in testing carried out by Dunn et al. (2005) and Konda (2003).

Live loading carried out in projects by Dunn et al. (2005), Konda (2003), and Van Erp et al. (2002) was noted, with some differences being truck type, size and speed. These may have been due to site specific reasons. Field testing for this project will be a combination of testing from these previous studies.

2.4. Analysis

According to Jenkins (2004), grillage analysis remains the standard procedure for analysing most beam and slab structures. It would seem a good idea to determine if the project bridge can be analysed using this method, however as Jenkins (2004) also states, there are various drawbacks in using grillage analysis.

Advantages: quick, standardised, easier to model.

Disadvantages: failure to deal with certain aspects of bridge behaviour, does not take into account construction method, generally conservative.

Tan et al. (1998) also concluded that grillage analysis was the most suitable model for bridge analysis due to its simplicity, accuracy and speed.

There is also debate over whether to use linear or non-linear finite element analysis when analysing composite structures. Lowe (1999), comments that linear analysis may produce overly conservative estimates, since it does not take plastic behaviour and global failure mechanisms into account. With the increase in computer size and power over the last few years (Jenkins, 2004), the additional time taken to run a non-linear analysis has been reduced, so this is essentially a non-issue in that non-linear analysis will tend to give the most accurate results.

Konda (2003) found that using a simply supported grillage model gave similar predictions of deflection values to those calculated theoretically, both of which overestimated when compared to field test results. As previously stated, grillage analysis tends to be conservative in predicting deflections, which could explain these results.

The effect of dynamic response of the bridge to heavy vehicles has been considered, but as the Austroads Publication AP-T23 (2003) suggests, this is a complex interaction between the bridge, the vehicles crossing the bridge, and the road profile. As such, it was deemed to be not necessary to be investigated in detail as it was outside the scope of the project, but mentioned as a possible factor for future research.

2.5. Further Study

As shown by the Australian Government publication Project Number PN05.2023 (2006), there is a great deal of interest in determining the viability of using hybrid and/or full FRP beams as replacement for existing hardwood timber beams in bridge construction. It is not only physical testing that must be undertaken, but also cost-benefit analysis on a whole-of-life scale that will determine future viability in the bridge construction industry.

The first Australian Standard for bridge design was published in 2004 (AS 51002004), but only covers the specific application of concrete, steel, and composite steel/concrete construction. Further research should be undertaken to include FRP material properties and standards.

As mentioned previously, the dynamic response of the bridge to heavy vehicles is another area where further study will improve the understanding of the complex interaction, with a large number of variables involved in the analysis of the dynamic response.

A "worldwide" survey of universities with FRP courses incorporated into their civil/structural engineering programmes (postgraduate and/or undergraduate) was undertaken by Mirmiran et al. (2003). This survey, however, neglected to include Australia, which appears to be quite an oversight as a number of Australian universities offer courses in FRPs. As this is not in the scope of the project, it could not really be taken further here, but could possibly be the starting point for another project.

The reason for including the above paper in this literature review is to highlight the need for continuing research into FRPs to ultimately lead to standards and specifications for the manufacturing and utilisation of fibre composites in more industries. If more universities cater for this type of research, the likelihood of standards becoming available should be increased.

2.6. Summary

The literature review highlighted the lack of standards and specifications in the manufacture of fibre composite products used in construction of structures, specifically bridges. This appears to a real issue, as the availability of such standards could increase the production of fibre composite products, which would decrease the production costs to the point where it should be economically viable to not only use these products for rehabilitation of existing bridges, but eventually replace timber, concrete and steel as the primary materials for construction of new bridges.

As far as field testing was concerned, it appears that the general method of testing remains consistent. There are few variables, some of which are: truck speed, truck size, number of instruments, types of instruments used (e.g. external versus internal strain gauges), and size of the bridge.

Grillage analysis tends to be the most used method, with the increased size and speed of computers making non-linear analysis much quicker and usually more accurate than linear analysis. However, grillage analysis has historically been shown to be relatively conservative when comparing theoretical analysis with experimental analysis.

3. METHODOLOGY

3.1. Test Bridge

The half-scale test bridge was constructed at the northwest end of Handley Street on USQ grounds in Toowoomba, and was constructed as a girder-deck composite bridge. It has a span of ten metres and is five metres wide. There are five 400 mm square girders bolted to concrete abutments, with 250 mm by 400 mm transverse stiffeners (diaphragms) bolted to the girders at 2500 mm intervals.

The deck is made up of eight main panels which are 1200 mm wide and approximately 120 mm deep. The deck panels are bolted and glued to the girders, with the two deck panels at the USQ end of the bridge glued with Sikaflex to create a flexible bond. All other deck panels are glued to create rigid bonds with the girders.

3.2. Testing Plan

Instrumentation selection was determined by what was available for use from the CEEFC and the Faculty of Engineering.

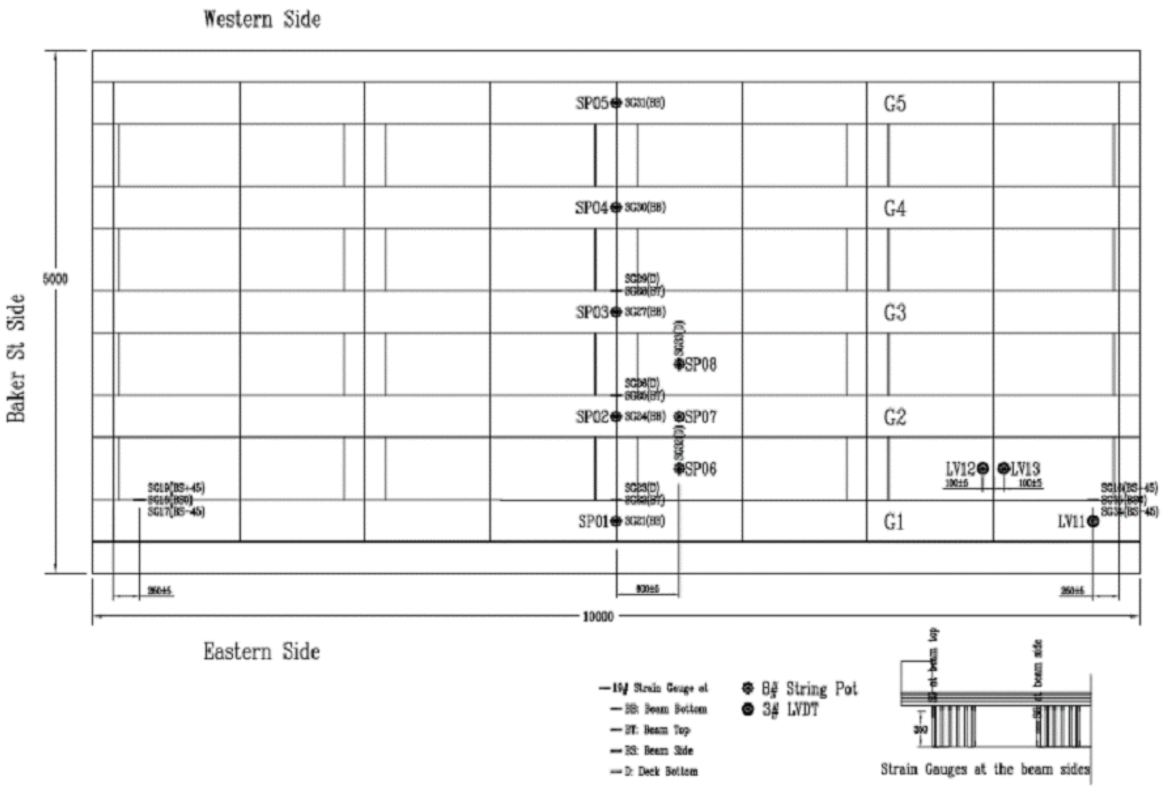

Figure 3-1: Instrumentation Positioning (Omar, T., 2007)

The use of three System 5000 monitoring systems, with a maximum capacity of 15 displacement gauge and 45 strain gauge channels, determined the maximum number of instruments to be installed. Leads and plugs needed to be soldered on to the instruments prior to installation.

Figure 3-2: System 5000 setup

3.3. Displacement Measurement

Five UniMeasure PA-15 string pot displacement gauges measured deflections at mid-span of the beams, and three PA-15 string pot displacement gauges were positioned to measure differential displacement between the deck panel and girder 2 (G2). The positioning of the three string pots to measure the differential girder-deck displacement can be seen in Figure 4-5.

Two Midori Precisions LP-50FB LVDT displacement transducers measured any differential deflection between adjacent deck units. Another LP-50 FB LVDT displacement transducer was positioned under girder 1 (G1) near the abutment of the bridge to measure displacement.

Figure 3-3: String pot positioning at girder mid-span

Figure 3-4: LVDT positioning for differential deck displacement

3.4. Strain Measurement

Kyowa KFG-20-120-C1-11L1M2R strain gauges measured flexural (tensile and compressive) strain and shear strain (delta rosette configuration) at selected positions on the beams and deck. Flexural strain gauges were positioned on the bottom of all girders (G1-5), 50 mm below the top of girders 1,2 and 3 (G1-3), and longitudinally on the deck at mid-span near girders 1,2 and 3 (G1-3). Transverse strain gauges were positioned on the deck near mid-span, and between Girders 1 and 2 (G1-2), and Girders 2 and 3 (G2-3).

Delta rosette configured strain gauges (-45°, 0°, +45°) were positioned at either end of Girder 1 (G1), as close as practicable to the abutments, to develop shear strain values as close as possible to maximum based on the recorded strains.

Figure 3-5: Delta rosette strain gauge configuration

All of the instruments were installed after construction was completed.

3.5. Loading

3.5.1. Initial Trial

An initial trial run was conducted using a small truck with an estimated 30 kN load on the rear axle. This run was used to get a general idea of the deflections and strains that could be expected in the formal testing. There were three string pots and two strain gauges installed on the bridge to measure deflections on girders 1, 2 and 3, and strains at the top and bottom of girder 1 to measure tensile and compressive girder strains.

Maximum mid-span deflections and maximum tensile and compressive strains were found from the data collected, and approximate values predicted for a rear axle load of 200 kN. The predicted values could then be compared with the measured values when the formal testing was completed and data available.

3.5.2. Truck Loads

The loading of the bridge occurred in two phases; in the first phase the truck was loaded to legal limit on the rear axle combination (166.5 kN), and in the second phase, four concrete blocks were added to the truck immediately above the rear axles to give an approximate axle combination load of 204.64 kN.

Figure 3-6: Phase 1 Loading

Figure 3-7: Phase 2 Loading

3.5.3. Run Pattern

The run pattern was determined by the need to load individual girders on the outside edges of the bridge (G1, G5) and centre girder (G3) as much as possible to create maximum deflections of these girders. This load pattern is indicative of a bridge in everyday use. Girder runs were marked on the bridge with whiter paint, and between-girder runs marked with yellow paint, as can be seen in Figures 9 and 10 above.

Increasing the speed of the truck to approximately 40 km/h for centre runs was used to see if there was any significant difference in displacement between crawl speed and the higher speed. Higher speed runs were not used on the outside edge girders due to safety reasons.

Table 3-1: Run Pattern

Run Ref #	Run Description
0	Initial runs
1	Truck RHS over G1
2	Truck RHS at the centreline between G1 and G2
3	Truck RHS over G2
4	Truck RHS at the centreline between G2 and G3
5	Truck down the centre of the bridge
6	Truck speeds down the centre of the bridge (~40 km/h)
7	Truck RHS over G3 (Not used)
8	Truck LHS over G5
9	Static testing

3.6. Analysis

Both field data analysis and finite element analysis were undertaken and results compared to determine the viability of using a simple grillage model to accurately predict deflections and strains in future construction of bridges. The field data analysis is covered in Chapter 4 and the finite element analysis is covered in Chapter 5, with analysis comparisons being discussed in Chapter 6.

3.7. Summary

The methodology set out above was followed and results obtained and analysed. Instrumentation was placed appropriately, and loading and run patterns were performed appropriately. Field data analysis and finite element analysis were undertaken, and results from both of the analyses compared and results discussed.

4. FIELD TEST RESULTS

4.1. Introduction

Once field testing was completed, recorded measurements were analysed using Microsoft Excel to determine: maximum deflection, deflection at abutments, maximum tensile, compressive and biaxial shear strains in the beams, maximum deflection of the deck, differential displacement between deck and girder, and any differential deflection between adjacent deck units.

From the mid-span deflection results, critical runs were found by using the maximum deflections in each of the girders as the determining factor.

Biaxial shear strains were calculated using the formula for strain (ε) along a line at an angle θ to the x-axis direction:

$$\varepsilon(\theta) = \varepsilon_x \cos^2\theta + \varepsilon_y \sin^2\theta + \gamma_{xy} \sin\theta\cos\theta \qquad \dots (1)$$

From equation 1, solving simultaneous equations for the three strain readings (-45°, 0°, +45°) gave the principal strains ε_x and ε_y, and the shear strain γ_{xy}.

Other analyses of the results were undertaken to determine the strain across the bridge at mid-span and maximum deflection, the strain through the girders to the deck in girders 1, 2 and 3, and deflection-load comparisons using the deflections from phases 1 and 2. All of these analyses were undertaken using data from the critical runs previously determined.

Both dynamic and static loading was taken into account. All field data collected and analysed can be found in Appendix D.

4.2. Preliminary Testing

As explained in Chapter 3, preliminary testing was conducted using a small truck with an estimated rear axle load of 30 kN. Table 3-1 shows the measured maximum mid-span values of deflections of girders 1, 2 and 3, and top and bottom strains at mid-span of girder 1. Data collected and plots developed can be found in Appendix D.

Figure 4-1: Initial truck trial

Table 4-1: Initial truck trial measurements and formal test predictions

Instrument & Position	Measured Maximum Value	Predicted Value for 200 kN (~ 6.5 x load)
String pot Girder 1 (mm)	4.55	~ 30
String pot Girder 2 (mm)	3.22	~ 20
String pot Girder 3 (mm)	2.35	~ 14
Strain Gauge bottom of Girder 1 (μ)	106	~ 650
Strain Gauge top of Girder 1 (μ)	-53	~ 325

4.3. Deflections

The analysis of mid-span deflections was used to determine critical runs for each of the girders. Further analysis of deflections included differential girder-deck deflection, differential deck deflection, girder deflections near the abutments, and load-deflection comparisons between phase 1 loading and phase 2 loading. Static load deflections were considered separately to dynamic load deflections. Graphs shown are representative of all results analysed, and all other graphs can be found in Appendix D.

4.3.1. Dynamic Deflections

4.3.1.1. Mid-Span Girder Deflections

The analysis of the mid-span girder deflections measured from the overloaded truck (Load phase 2) runs have been used to determine the critical runs, so that further testing time can be minimised by discarding unnecessary truck runs.

From the mid-span deflection analysis as shown in the graphs below, the following runs have been confirmed as critical for each girder:

Girder 1: Run 1
Girder 2: Run 1
Girder 3: Run 5 and Run 6
Girder 4: Run 8
Girder 5: Run 8

These results make sense, as the largest deflections on each girder occur when the majority of the truck weight is directly above them. In each graph, the zero point on the x axis is taken where the rearmost axle arrives on the bridge. As the distance from front axle to rearmost axle is six metres, deflection of the girders will commence at -6 metres and cease at 10 metres, when the rear axle moves off the far end of the bridge.

As all runs were at slightly different speeds, factors have been applied to rescale each run so that deflection is initiated at -6 metres and ceases at 10 metres.

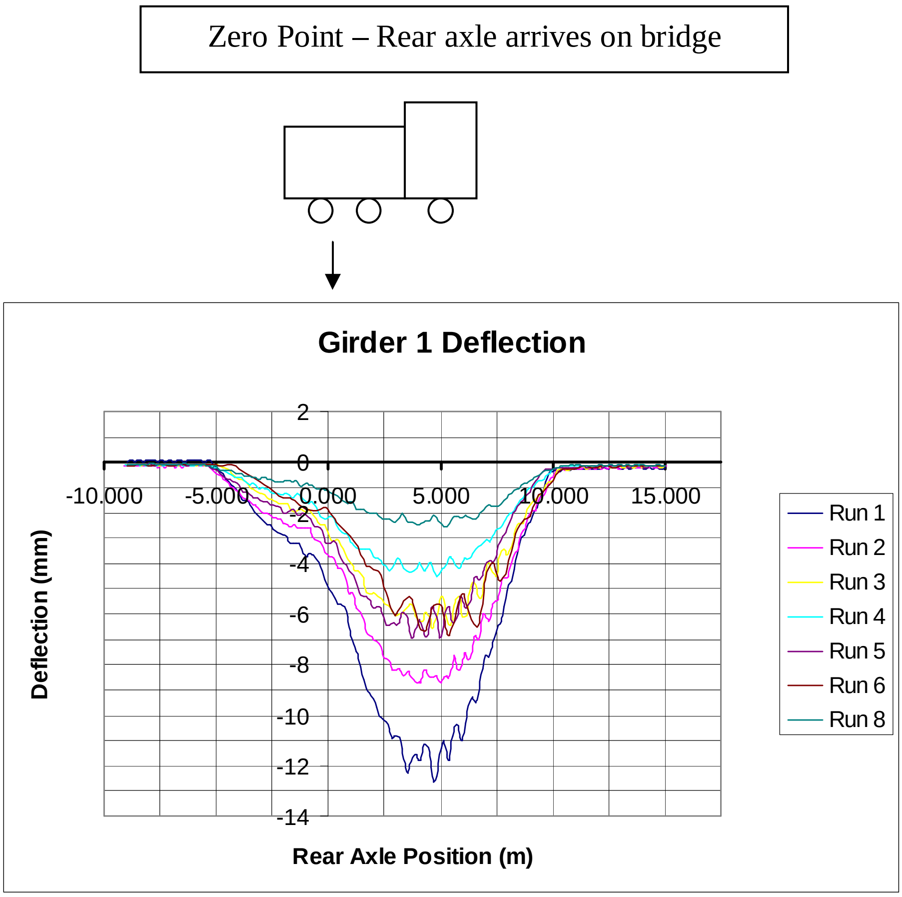

Figure 4-2: Girder 1 Mid-span Deflection

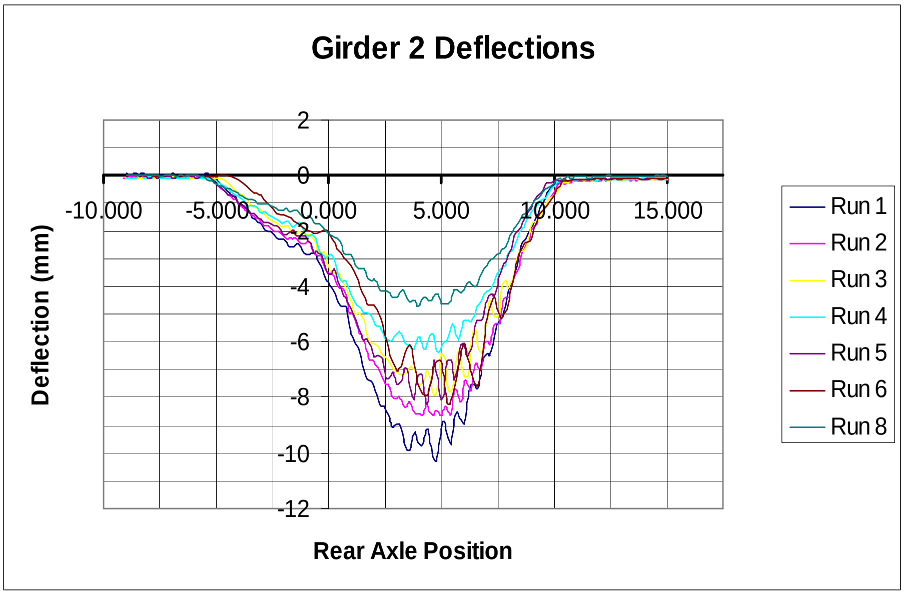

Figure 4-3: Girder 2 Mid-span Deflection

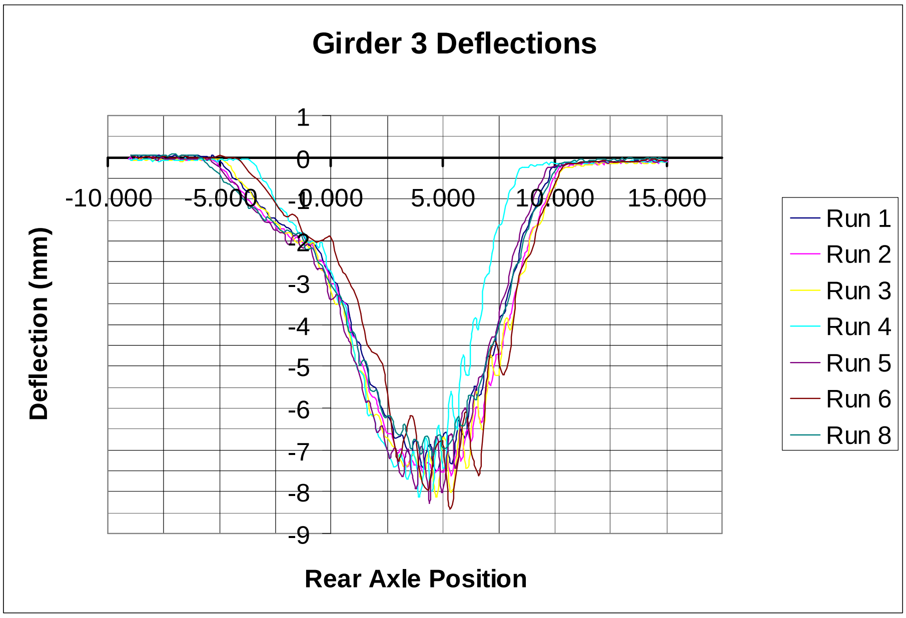

Figure 4-4: Girder 3 Mid-span Deflection

As girders 1 and 5 and girders 2 and 4 are similar, the mid-span deflection plots of girders 4 and 5 will not be shown here. They can be found in Appendix D with the data used to develop the plots.

Table 4-2: Critical Run and Maximum Mid-span Deflection of girders

Girder #	Run #	Max Deflection (mm)	Rear axle Position
1	1	12.633	~ 300 mm to left of mid-span
2	1	10.279	~ 300 mm to left of mid-span
3	5	8.289	~ 600 mm to left of mid-span
3	6	8.434	~ 600 mm to left of mid-span
4	8	8.968	~ 400 mm to left of mid-span
5	8	11.505	~ 400 mm to left of mid-span

As Table 4-2 shows, all of the maximum girder deflections are well below the design maximum limit of 20 mm. Once the critical runs were established, other analysis occurred.

4.3.1.2. Differential Girder-Deck Deflection

The positioning of instruments SP06 (deck), SP07 (girder) and SP08 (deck) was used to determine if there was considerable differential displacement between the deck panels and the girders that were epoxy glued together (rigid bond). The positioning is shown in Figure 4-5, below. The spacing between instruments was approximately 500 mm.

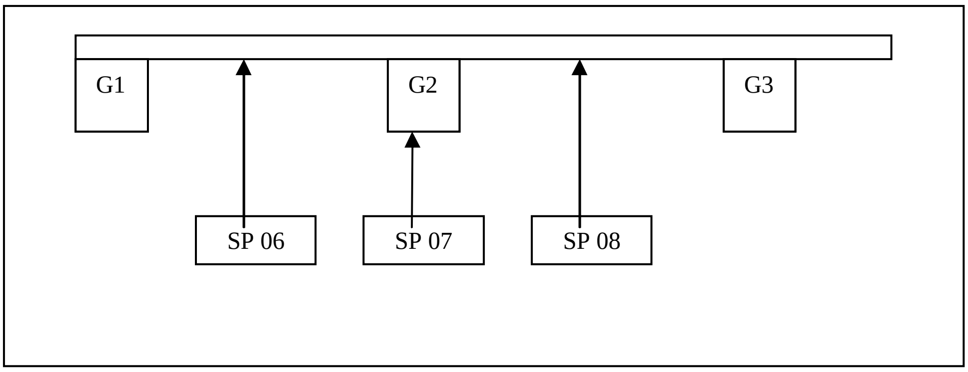

Figure 4-5: Placement of String Pots 6, 7 and 8

Figures 4-6 and 4-7 show that the displacement on runs 1 and 8 is virtually linear across the bridge (< 0.1 mm differential displacement), but the girder deflects sightly more than the deck in runs 5 and 6 (Figures 4-8 and 4-9). However, this differential displacement is approximately 0.2 mm (~ 0.04%), so should not be of major significance. Only critical runs were considered when plotting the results.

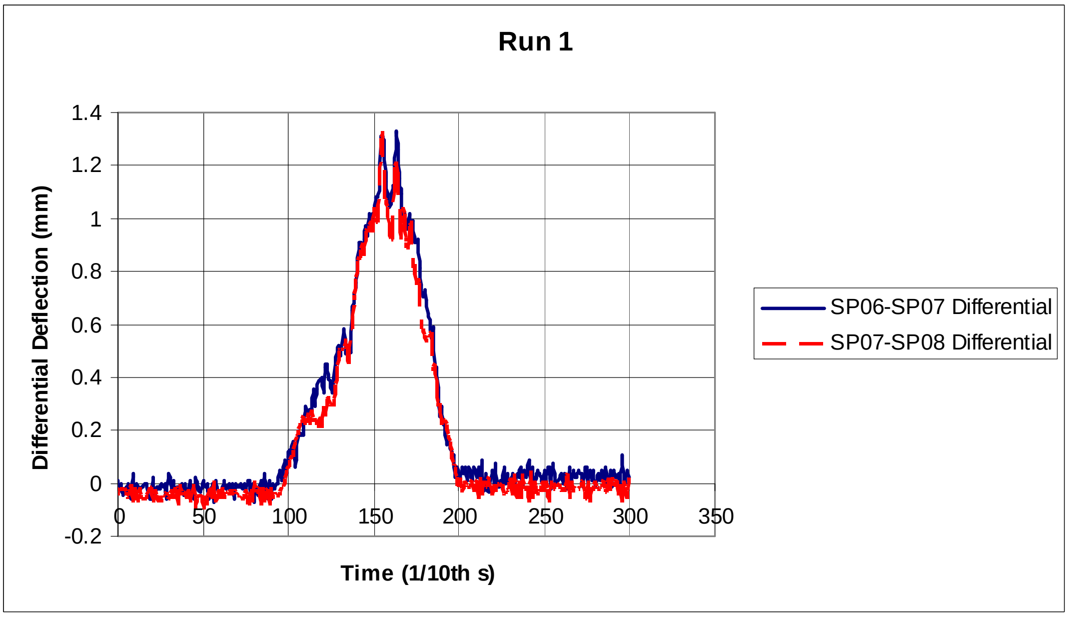

Figure 4-6: Differential Girder-Deck Deflection – Run 1

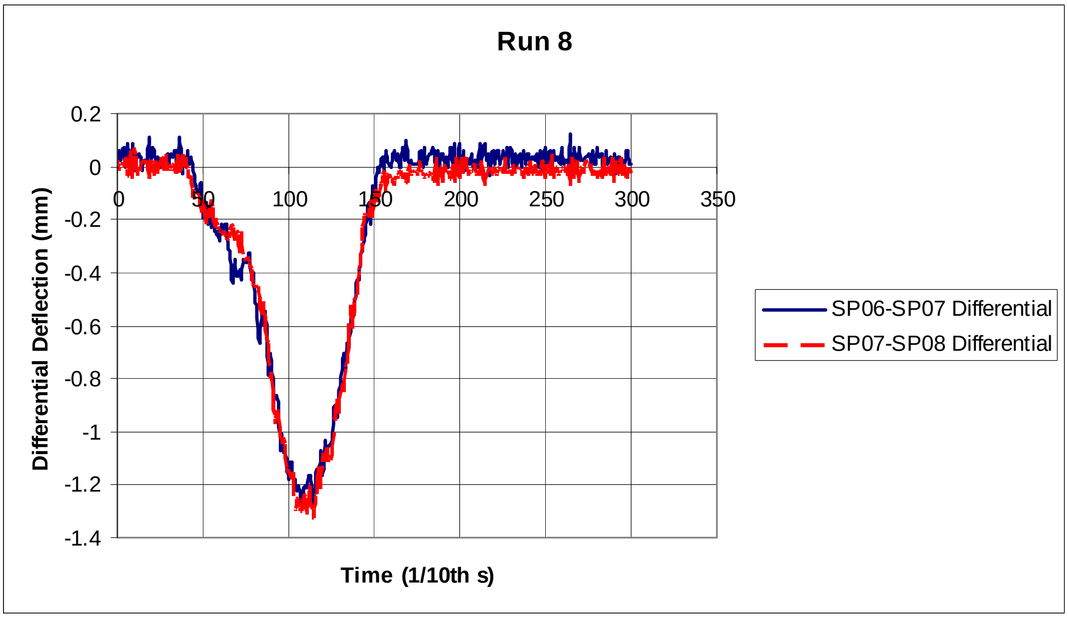

Figure 4-7: Differential Girder-Deck Deflection - Run 8

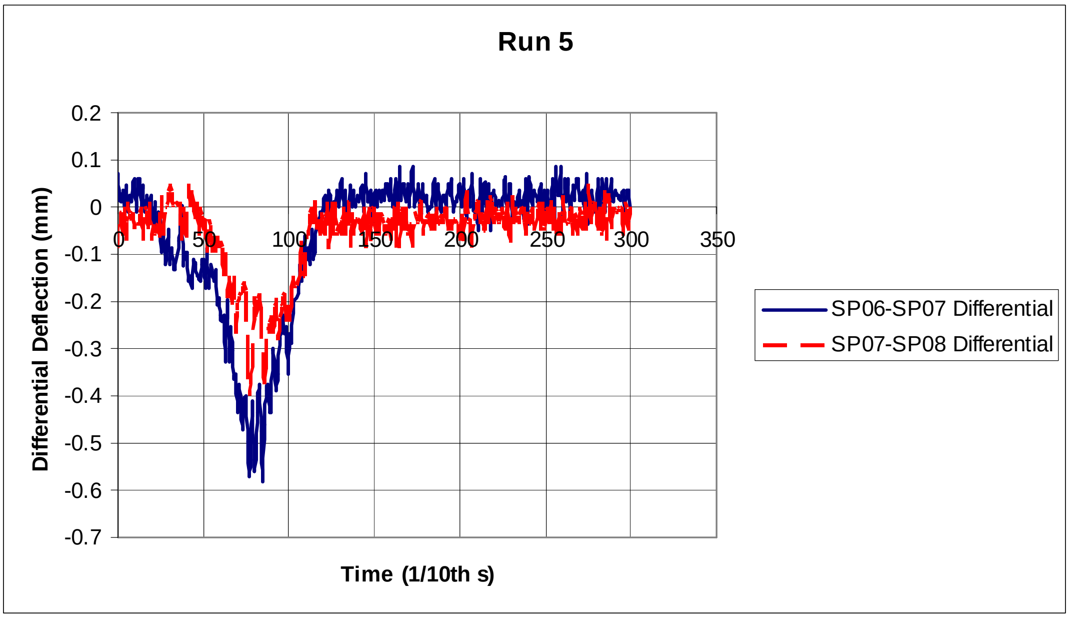

Figure 4-8: Differential Girder-Deck Deflection - Run 5

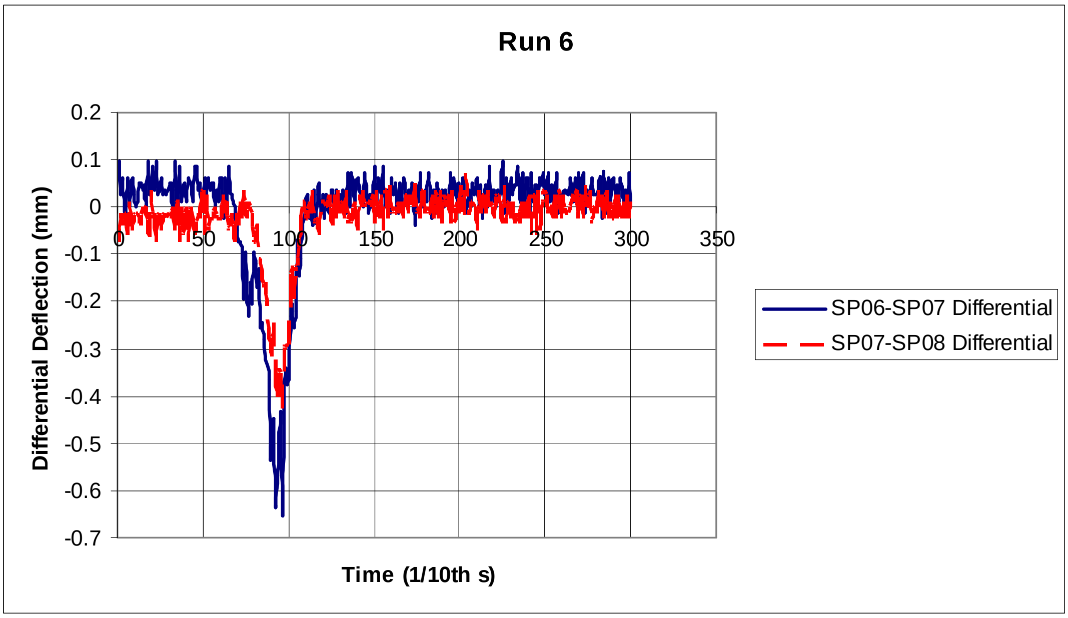

Figure 4-9: Differential Girder-Deck Deflection - Run 6

4.3.1.3. Differential Deck Deflection

As shown in Figure 4-10, the differential displacement measured (LV12, LV13) at the USQ end of the bridge, where the deck was glued to the girders with Sikaflex (flexible bond), reached a maximum of 0.7 mm (Run 1). The magnitude of this deflection (approximately 10-20%) may have some structural significance relative to the overall displacement occurring at the same time, and could be regarded as an influencing factor. Figure 4-11 shows the maximum differential displacement on each of the critical runs.

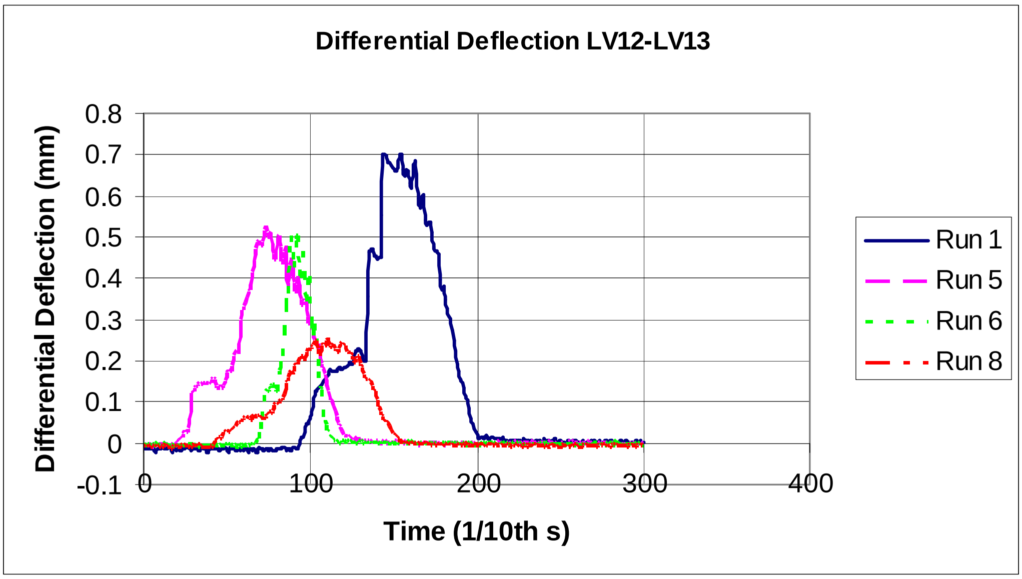

Figure 4-10: Differential Deck Deflections – Critical Runs

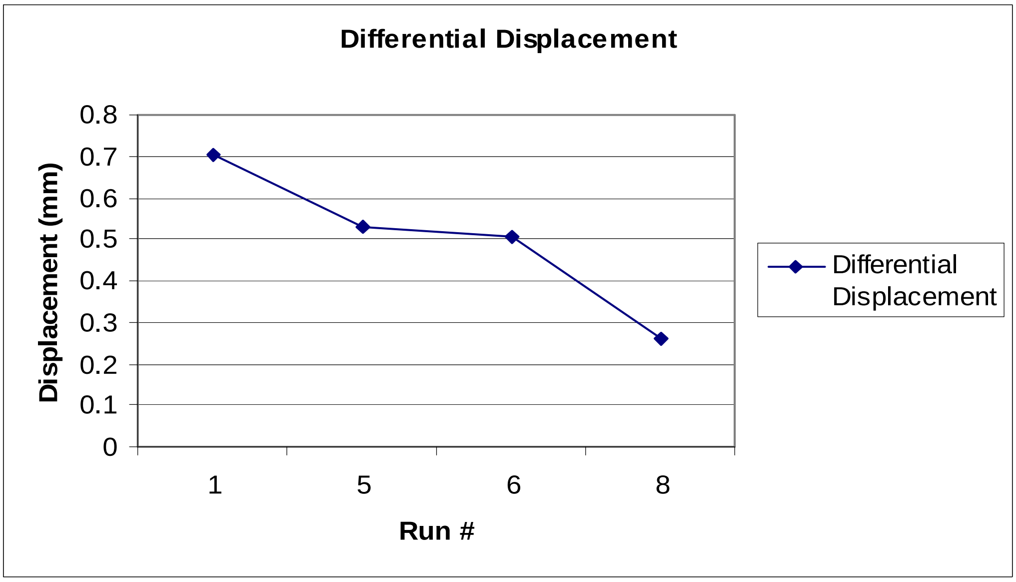

Figure 4-11: Maximum Differential Deck Displacement

4.3.1.4. Deflection near Abutments

This measurement was taken to use in comparison with the finite element model analysis, and does not have any significant structural bearing. As the number of measuring instruments was limited, readings were only taken from Girder 1. As with the differential girder-deck displacement, only critical runs were considered.

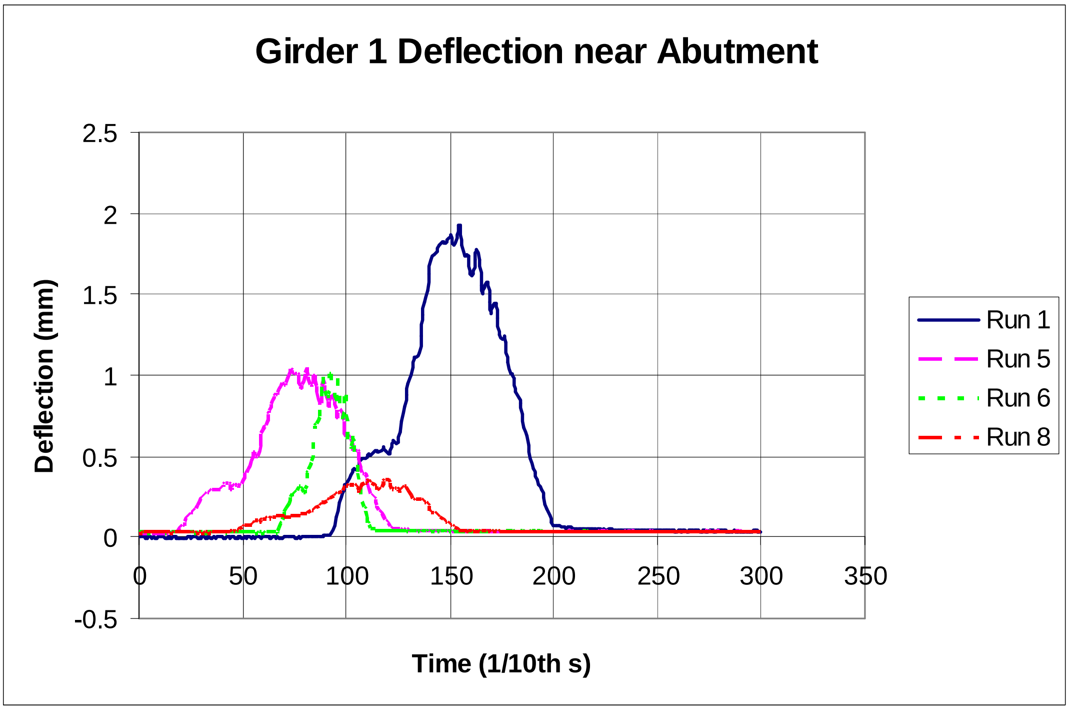

Figure 4-12: Deflection near Abutment at USQ end of Bridge

4.3.1.5. Load-Deflection Comparisons

The load-deflection plots appear to be reasonably linear when the load is carried by the appropriate girders for the particular runs i.e. Girders 1, 2 and 3 for Run 1, Girders 2, 3 and 4 from Runs 5 and 6, and Girders 3, 4 and 5 for Run 8. This is shown in Figures 4-13 to 4-16.

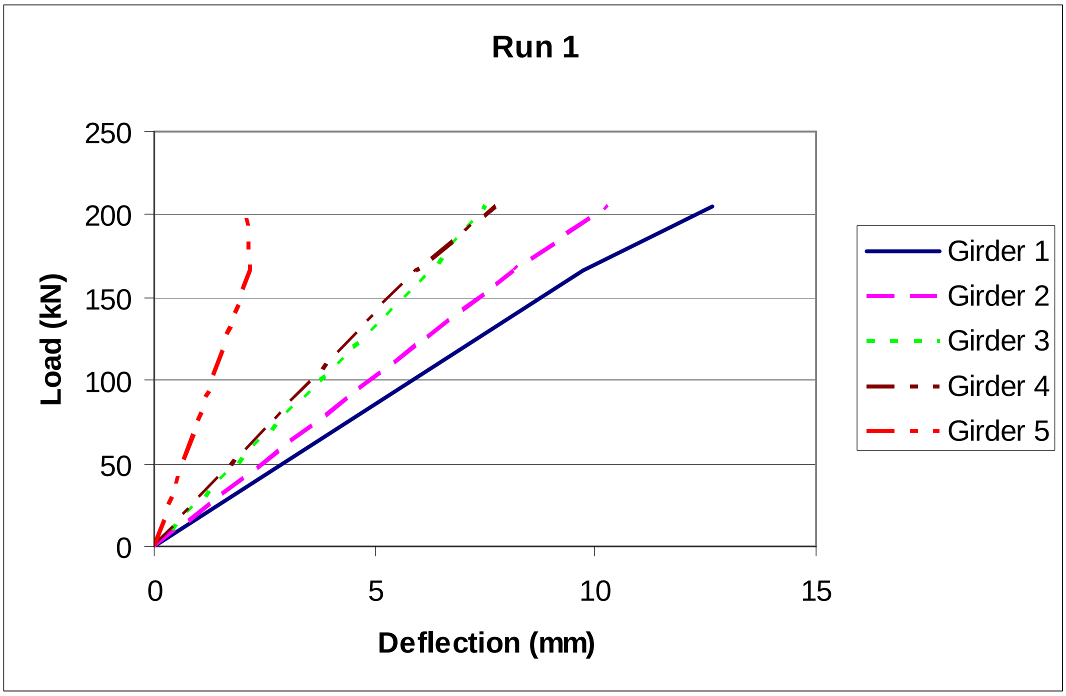

Figure 4-13: Load-Deflection Comparison - Run 1

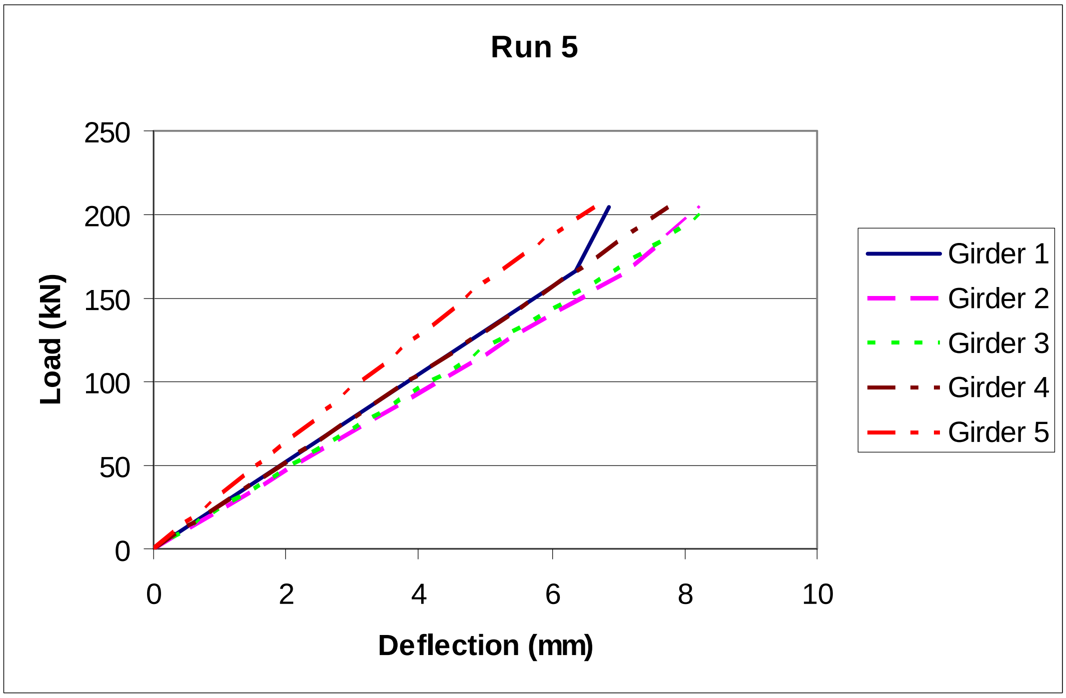

Figure 4-14: Load-Deflection Comparison - Run 5

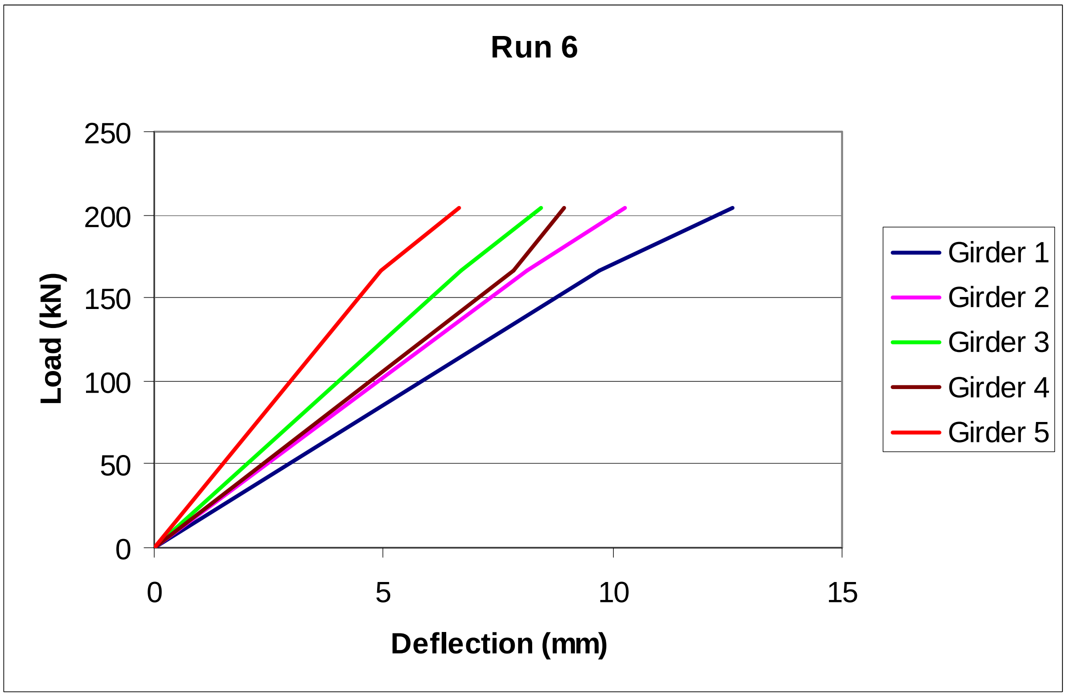

Figure 4-15: Load-Deflection Comparison - Run 6

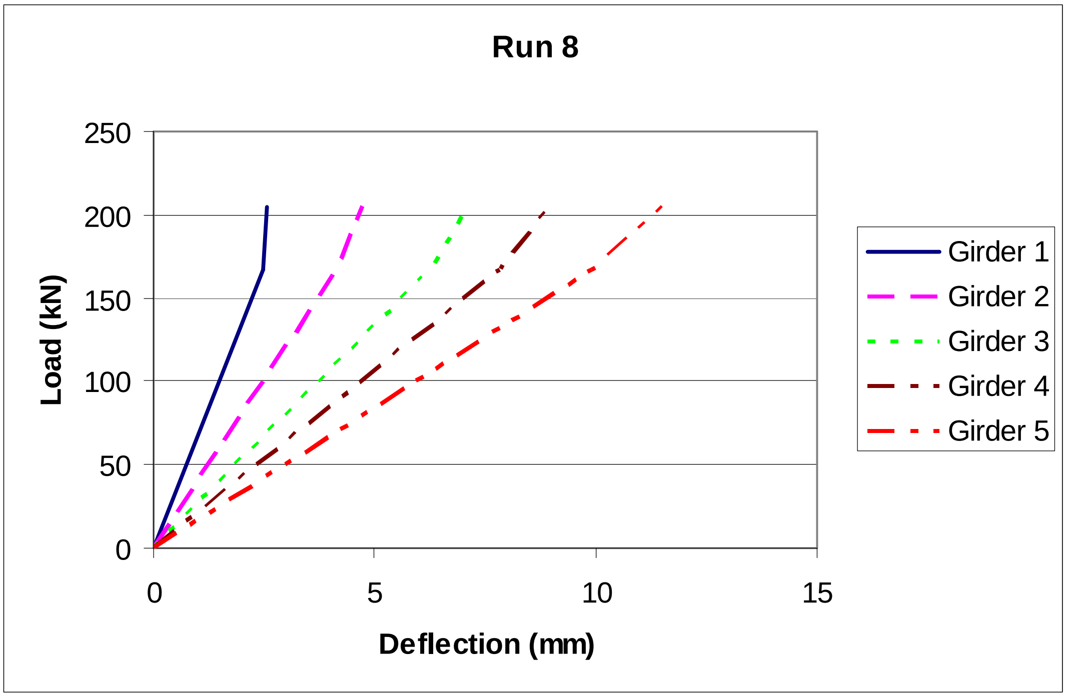

Figure 4-16: Load-Deflection Comparison - Run 8

4.3.2. Static Deflections

The analysis of static deflections and comparison with dynamic deflections previously analysed should give an indication of the extent of the dynamic response of the bridge to heavy vehicles. Vehicle bridge interaction (VBI) has been studied extensively and software has been developed to show that a large number of variables need to be taken into account when analysing field test results. This was outside the scope of this project, and consequently was not analysed. The following results show that there was some dynamic response of the bridge to the truck runs, particularly the higher speed run (Run 6), but no extensive analysis was undertaken.

4.3.2.1. Mid-span Girder Deflections

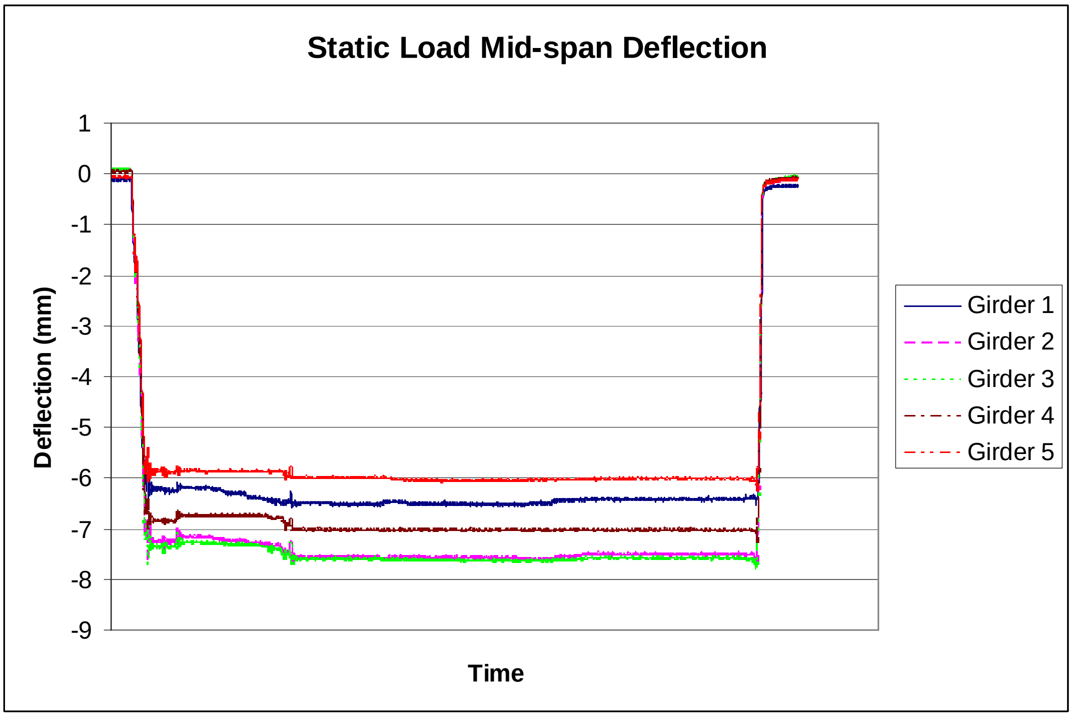

Figure 4-17: Static Load Mid-span Deflection

From Runs 5 and 6, the maximum dynamic load deflection of each girder can be taken from the data, and compared with the static load deflection, as shown in Table 4-3.

Table 4-3: Experimental Static-Dynamic Differential of Bridge

Girder #	Dynamic Deflection (mm)	Static Deflection (mm)	Differential (mm)
1	6.942	6.590	0.352
2	8.288	7.694	0.594
3	8.434	7.791	0.643
4	7.779	7.294	0.485
5	6.650	6.274	0.376

This can be seen more easily in Figure 4-18. This figure also shows that the central runs were actually run a little to the left of centre, and this may have slightly skewed the results. For future testing, it would be advisable to re-mark thee proposed wheel paths to run the truck equidistant from both edges, and remove any skewing of the data.

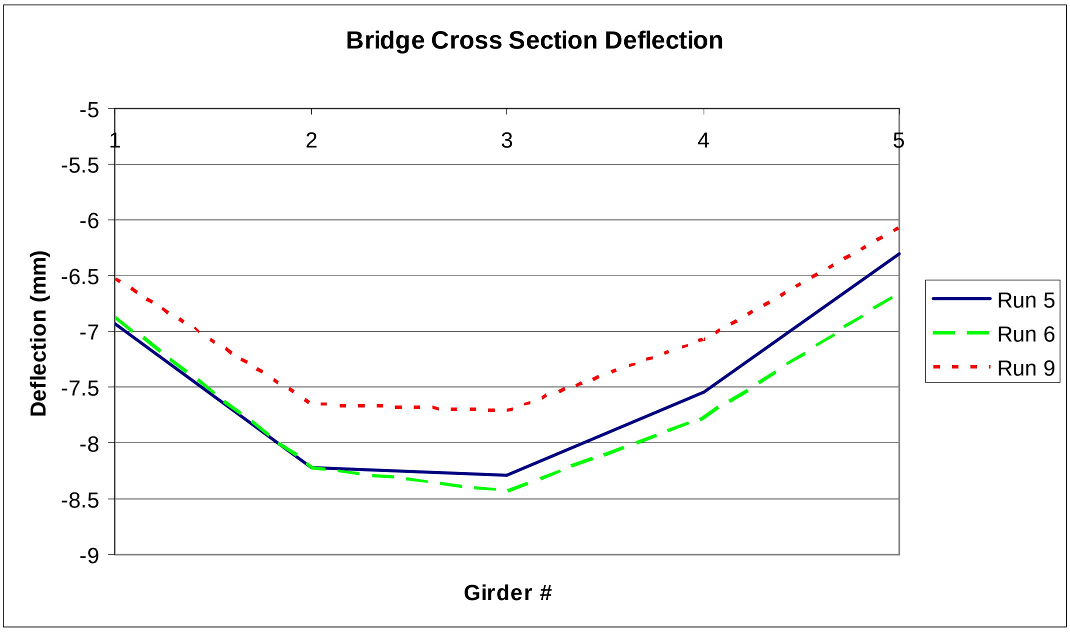

Figure 4-18: Bridge Cross Section under Central Runs

The magnitude of this dynamic response (< 1 mm) would appear to have little or no structural significance; however as this is outside the scope of this project, any further research of these results serves no particular purpose but may be undertaken in future study.

4.3.2.2. Differential Girder-Deck Deflection

As with the dynamic results, the static loading shows reasonable linearity across the deck-girder-deck instrument positioning. The differential deflection of approximately 0.2 mm suggests that the differential girder-deck deflection has minimal structural significance, and further testing need not include measurements taken from these instruments.

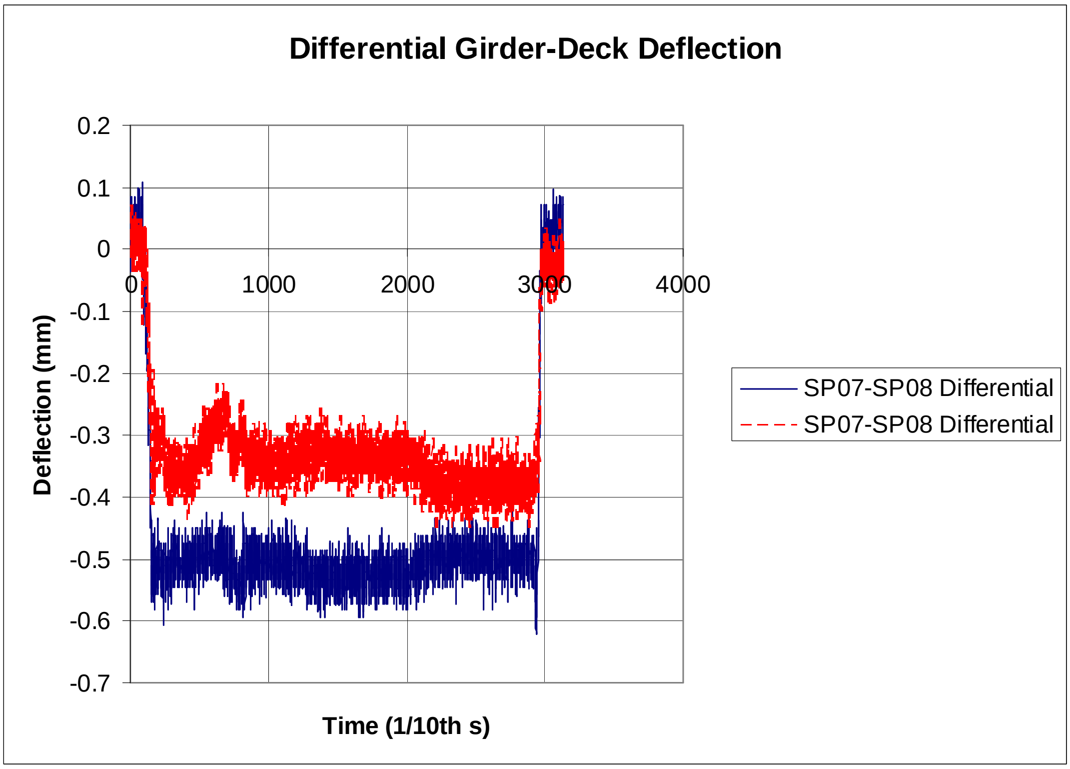

Figure 4-19: Differential Girder-Deck Deflection

4.3.2.3. Differential Deck Deflection

The static load differential deck deflection of 0.4 – 0.5 mm, as shown in Figure 4-20, is in close agreement to the dynamic load centre run differential deck deflections, particularly that of Run 6.

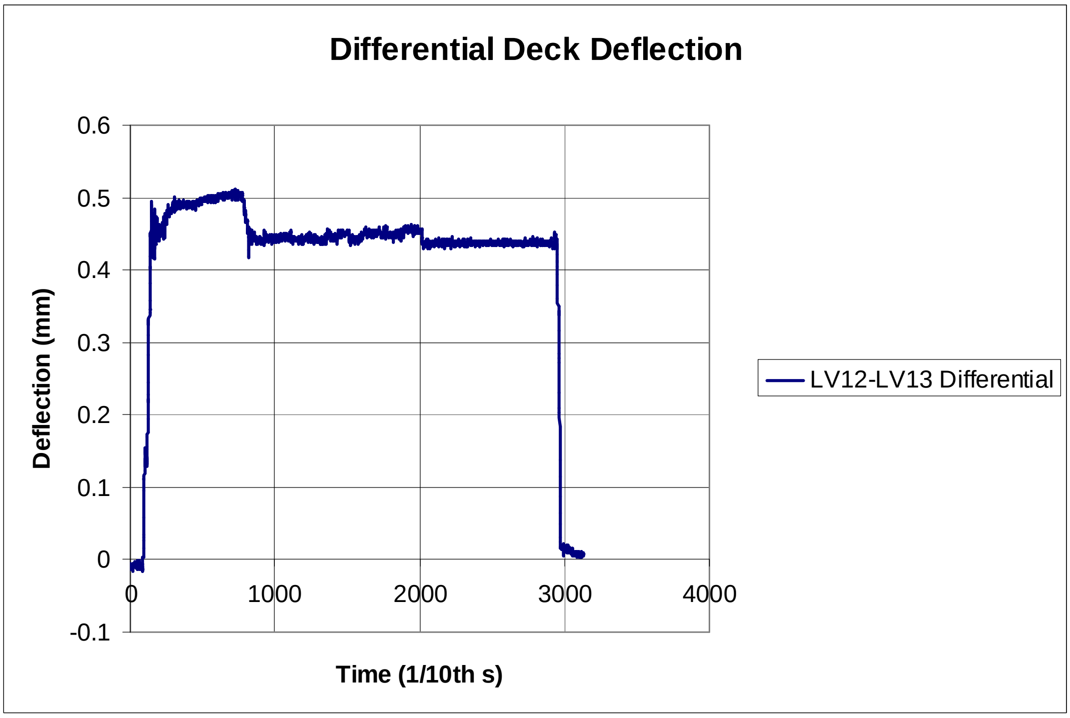

Figure 4-20: Differential Deck Deflection - Run 9

4.3.2.4. Deflection near Abutments

The average deflection of Girder 1 near the abutment at the USQ end of the bridge under static load was approximately 1 mm (Figure 4-21). This value is in close agreement with the dynamic deflection on Runs 5 and 6 of approximately 1 mm (Figure 4-12).

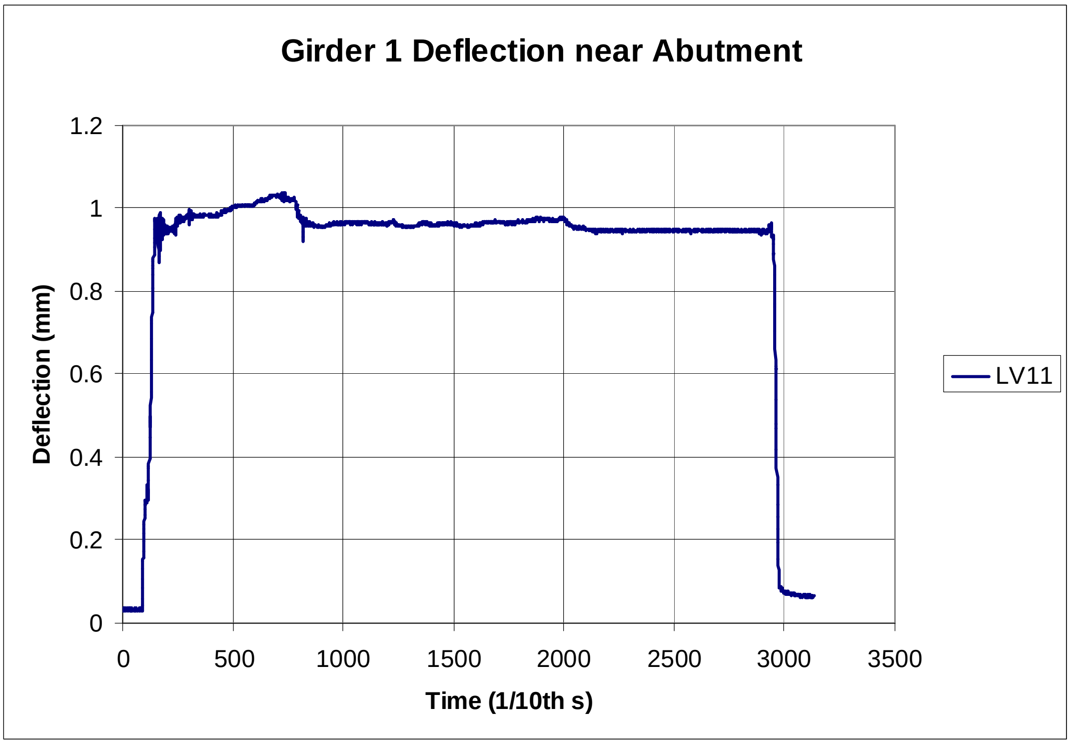

Figure 4-21: Deflection near Abutment at USQ end of Bridge

4.3.2.5. Load-Deflection Comparisons

As no static loading was undertaken in phase 1, no load-deflection comparisons were made. This area may be researched in more detail in future projects.

4.4. Strains

From the critical runs, strain readings were taken and analysed. Strain gauges were placed on the soffit of each girder (SG21, SG24, SG27, SG30 and SG31) to measure the maximum tensile strain in the girders, and 50 mm below the top of girders 1, 2 and 3 (SG22, SG25 and SG28) to measure the maximum compressive strain in the girders.

Strain gauges were also placed longitudinally (SG23, SG26 and SG29) on the underside of the deck at mid-span near girders 1, 2 and 3, and transverse (SG32 and SG33) near mid-span between girders 1 and 2, and girders 2 and 3.

Delta rosette configured strain gauges (-45°, 0°, +45°) were positioned at either end of girder 1, as close as practicable to the abutments, to record strains to be use din the calculation of shear strains as close as possible to maximum (SG34, SG35 and SG36 at the USQ end of the bridge, and SG37, SG38 and SG39 at the Handley St end of the bridge).

All strain values recorded were well below the yield and/or shear strain capacity (approximately 2500 μ for yield strain of steel reinforcing in girders, and approximately 12000 μ for glass fibre yield strain in deck units) of the bridge members.

4.4.1. Dynamic Loading Strains

The strain analyses were undertaken in a similar manner to the deflection analyses, with the dynamic and static load cases taken separately, then compared.

4.4.1.1. Girder Tensile Strains

The maximum tensile strain of 358 μ was recorded in girder 5 on Run 8. Figures 4-22 to 4-24 show the tensile strains recorded on each girder for the critical runs.

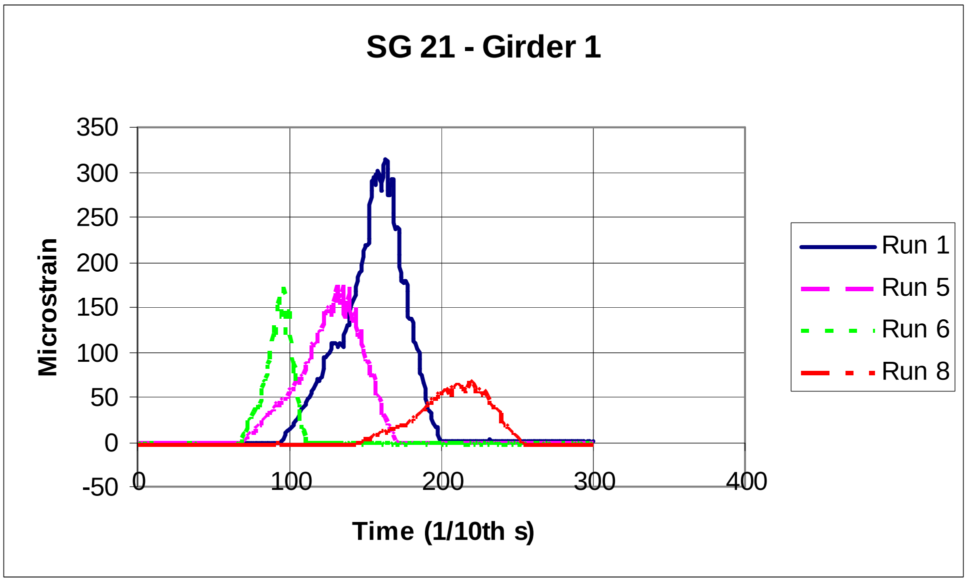

Figure 4-22: Tensile Strain - Girder 1

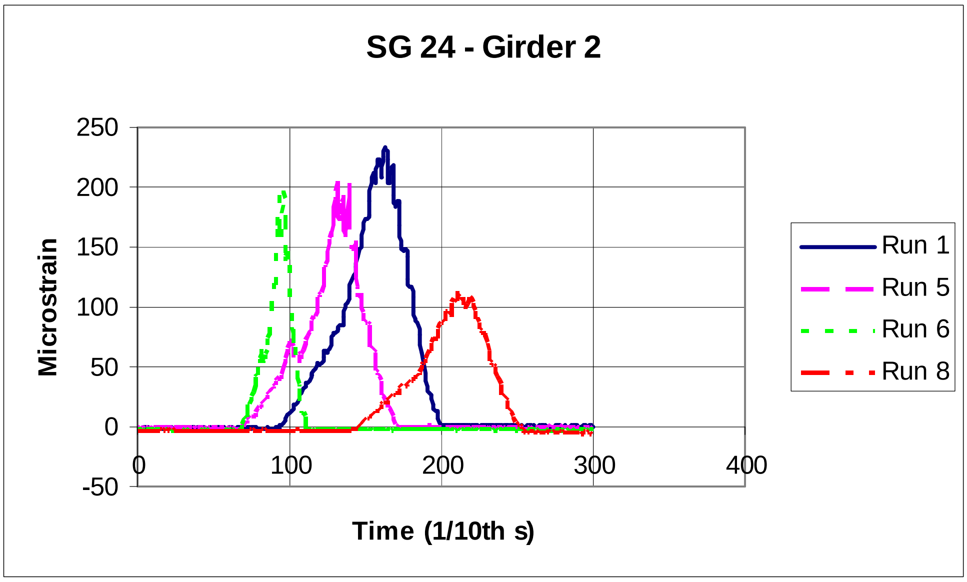

Figure 4-23: Tensile Strain - Girder 2

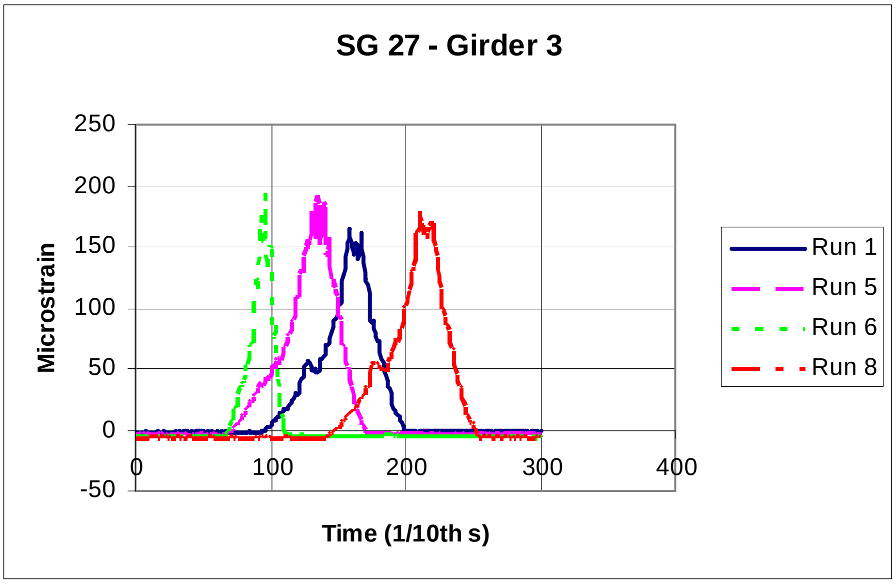

Figure 4-24: Tensile Strain - Girder 3

As with the mid-span deflections, girders 1 and 5 and girders 2 and 4 are similar, so the tensile strain plots of girders 4 and 5 will not be shown here. They can be found in Appendix D, together with the data used to develop the plots.

Table 4-4 shows the maximum tensile strains in all girders, and the runs associated with the maximum values.

Table 4-4: Maximum Girder Tensile Strains

Girder #	Maximum Tensile Strain (μ)	Run #
1	315	1
2	234	1
3	195	6
4	230	8
5	358	8

A plot of strain distribution at maximum deflection across the bridge was also developed (Figure 4-25) and inspected for linearity. As can be seen, all critical runs show a reasonable degree of linearity across the bridge.

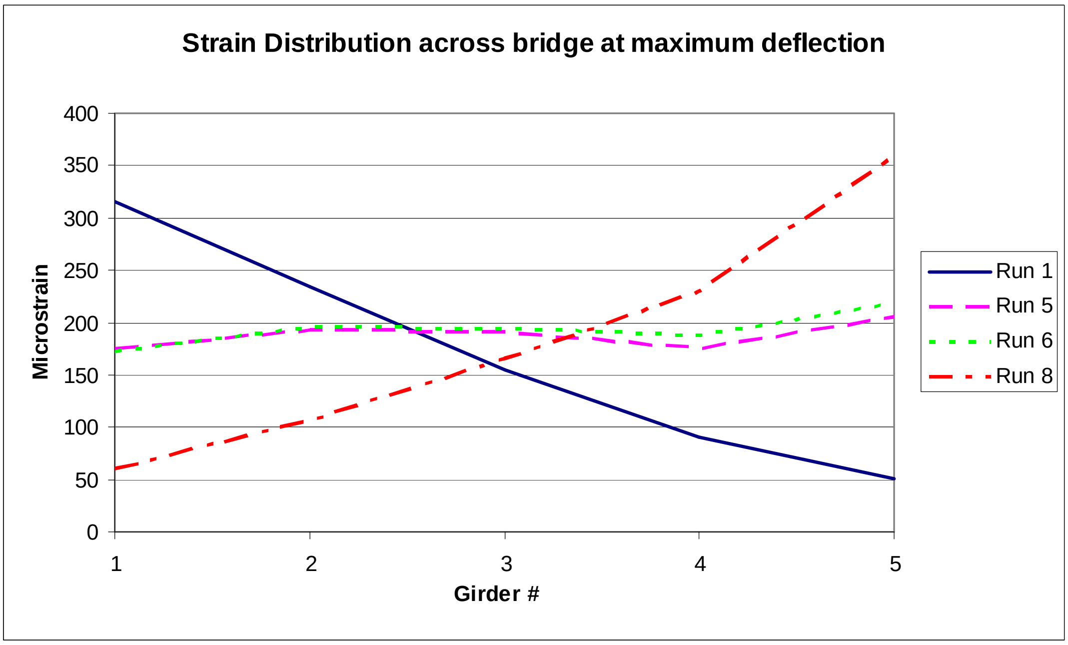

Figure 4-25: Strain Distribution across bridge at maximum deflection

4.4.1.2. Girder Compressive Strains

Only girders 1, 2 and 3 had measurements taken of the compressive strain near the top of the girders. The maximum compressive strain of 170 μ was recorded in girder 1 on Run 1. Figures 4-26 to 4-28 show the compressive strains recorded on each girder for the critical runs.

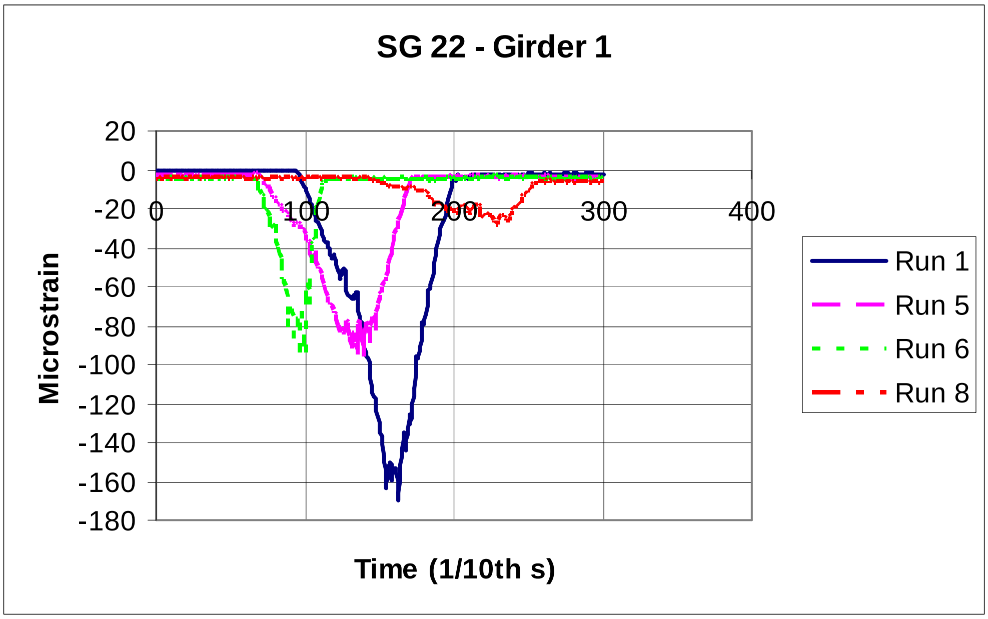

Figure 4-26: Compressive Strain - Girder 1

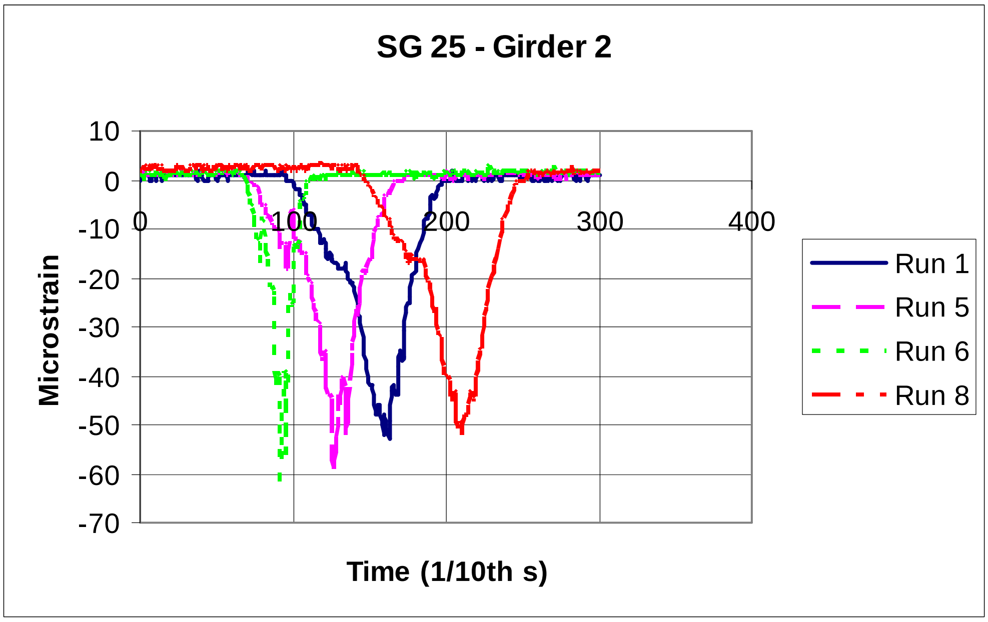

Figure 4-27: Compressive Strain - Girder 2

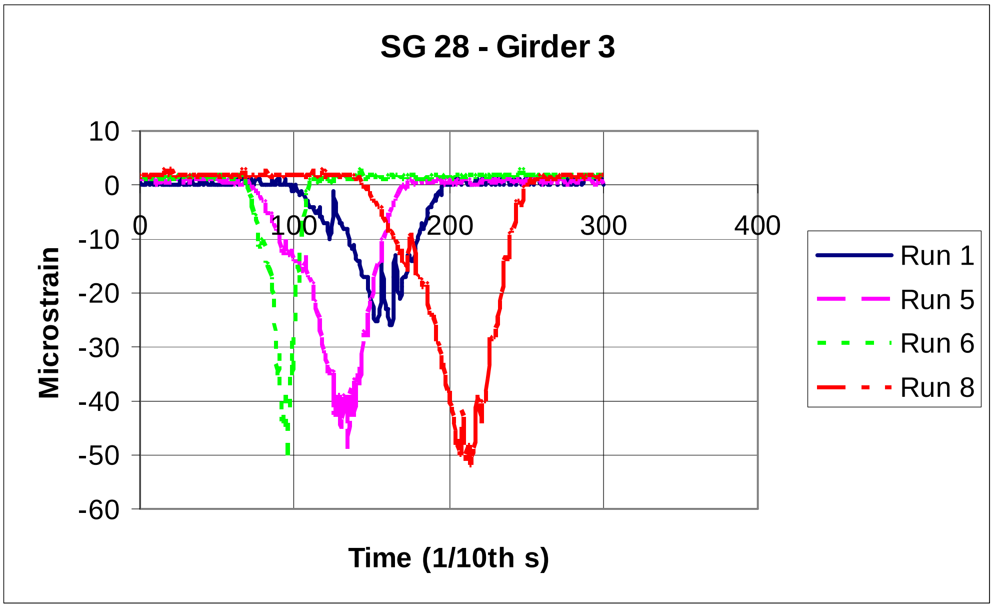

Figure 4-28: Compressive Strain - Girder 3

Table 4-5 shows the maximum compressive strains in the three girders, and the runs associated with the maximum values.

Table 4-5: Maximum Girder Compressive Strains

Girder #	Maximum Compressive Strain (μ)	Run #
1	170	1
2	61	6
3	52	8

4.4.1.3. Deck Strains

Both longitudinal (at mid-span) and transverse (near mid-span) strain were recorded.

Longitudinal Deck Strain

Measurements were taken on the underside of the deck at mid-span next to the top of girders 1, 2, and 3 (SG23, SG26 and SG29). The maximum compressive strain of 96 μ was recorded near girder 2 on Run 5, and the maximum tensile stress of 36 μ was recorded near girder 3 on Run 8. The tensile stresses may be due to the vibration of the bridge, and will not be analysed further. Figures 4-29 to 4-31 show the strains recorded near each girder for the critical runs.

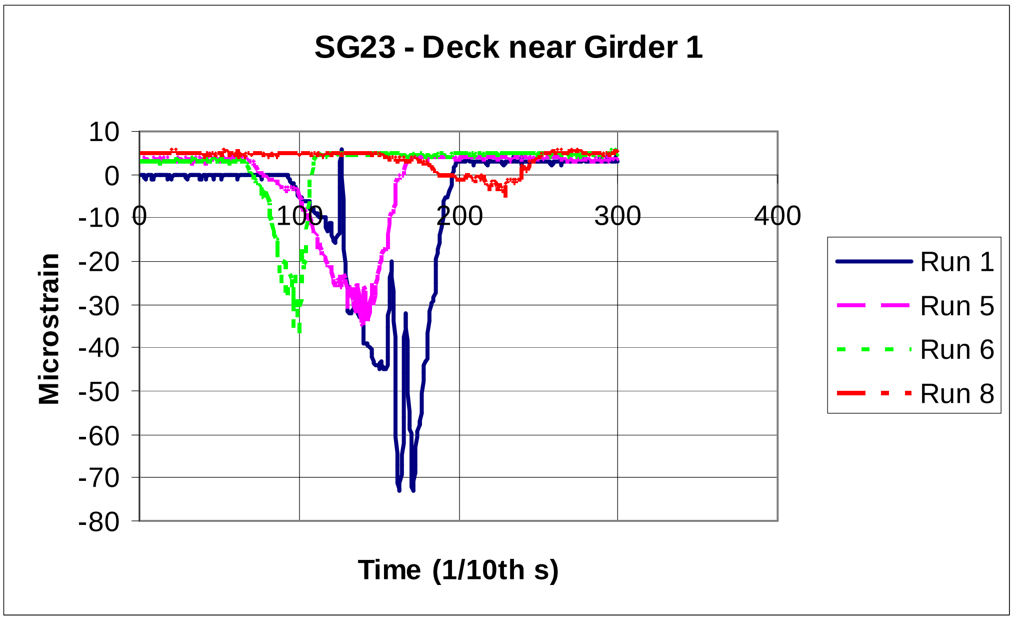

Figure 4-29: Longitudinal Strain on Deck near Girder 1

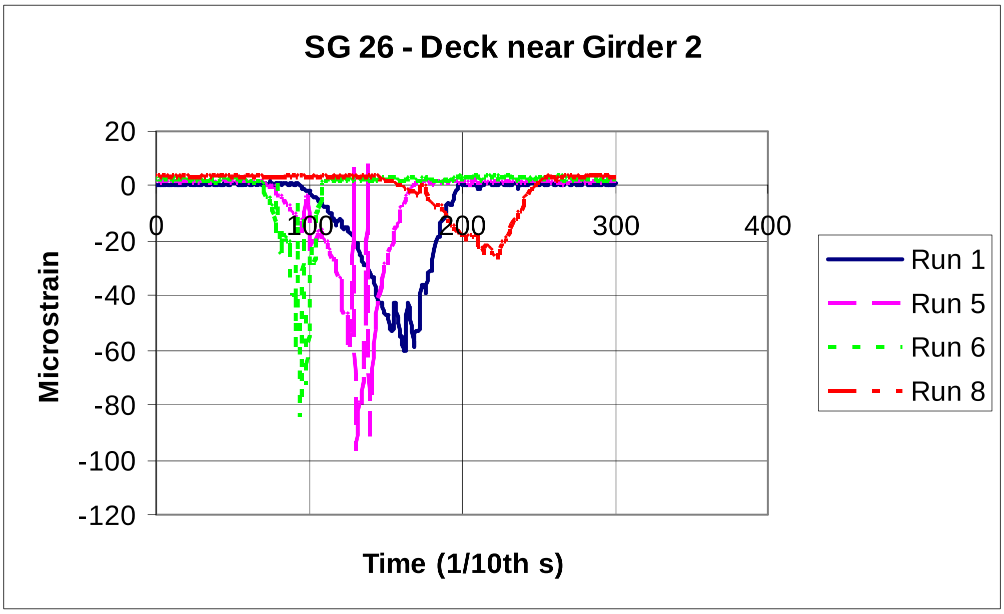

Figure 4-30: Longitudinal Strain on Deck near Girder 2

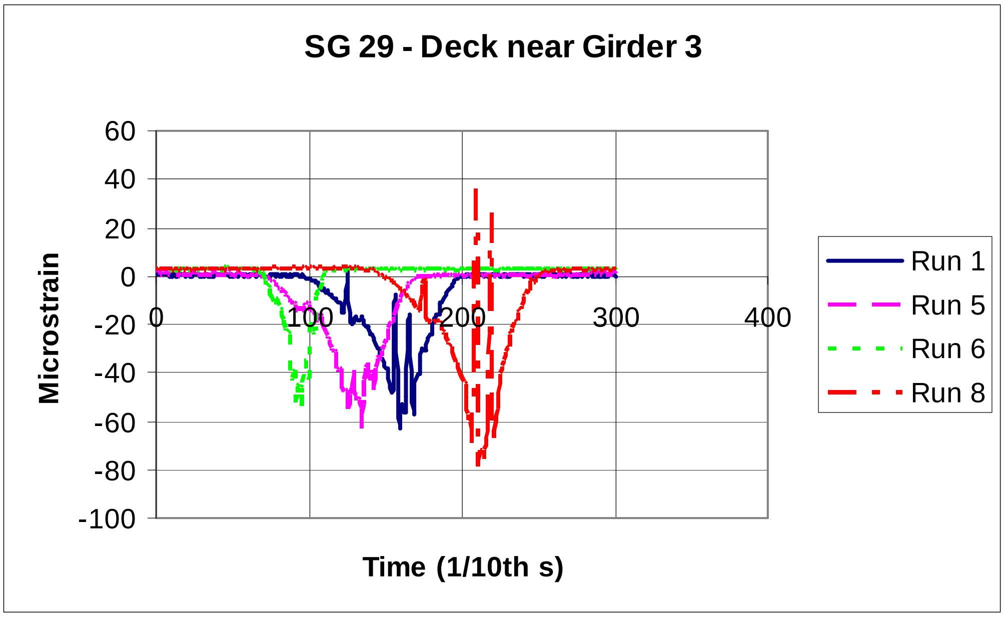

Figure 4-31: Longitudinal Strain on Deck near Girder 3

Table 4-6 shows the maximum compressive longitudinal strains in the underside of the deck panels near the three girders, and the runs associated with the maximum values.

Table 4-6: Maximum Deck Longitudinal Compressive Strains

Deck near Girder #	Maximum Compressive Strain (μ)	Run #
1	73	1
2	96	6
3	78	8

Transverse Deck Strains

Two measurements of transverse deck strain were taken near mid-span, between girders 1 and 2 (SG32), and between girders 2 and 3 (SG33). The maximum tensile strain of 276 μ was recorded on Run 6 (SG33), and the maximum compressive strain of 16 μ was recorded on Run 8 (SG32). Figures 4-32 and 4-33 show the strains recorded for the critical runs.

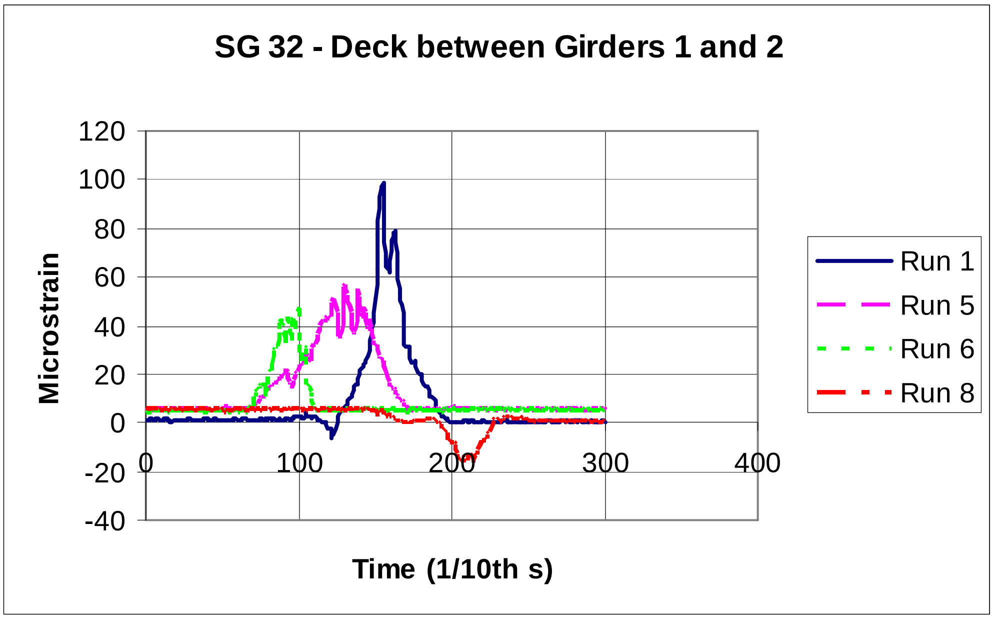

Figure 4-32: Transverse Deck Strain between girders 1 and 2

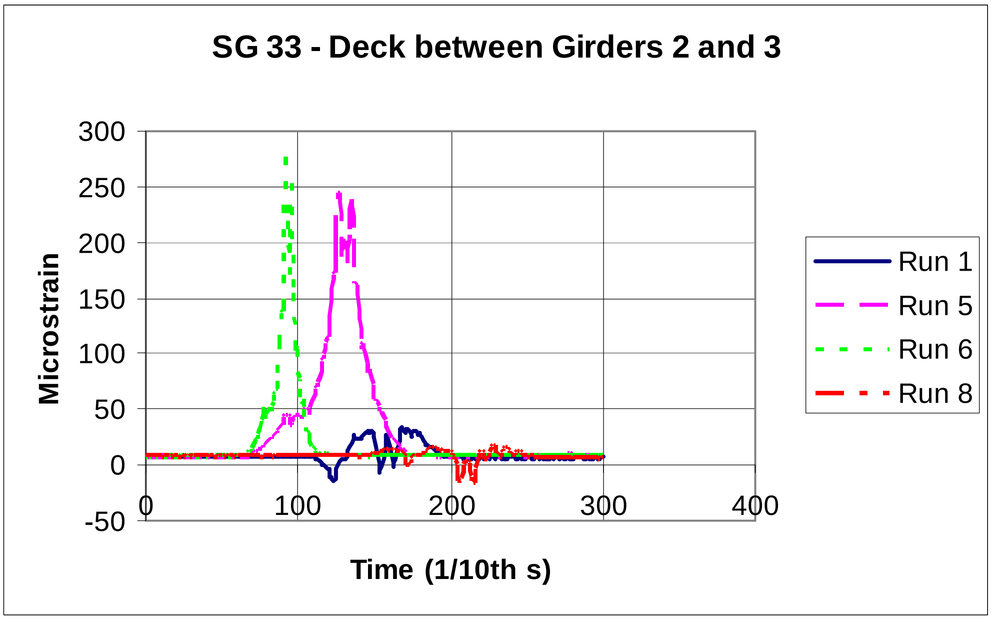

Figure 4-33: Transverse Strain on Deck between girders 2 and 3

Table 4-7 shows the maximum tensile and compressive transverse strains in the underside of the deck panels near the three girders, and the runs associated with the maximum values.

Table 4-7: Maximum Deck Transverse Strains

Strain Gauge #	Maximum Strain (μ)	Run #
SG32	99 (tensile)	1
SG32	16 (compressive)	8
SG33	276 (tensile)	6
SG33	15 (compressive)	8

4.4.1.4. Combination Strains

Using the readings taken from the girder tensile and compressive strains, and the longitudinal deck strains, combination strain plots were developed to show the strain through the girders to the deck for girders 1, 2 and 3. These plots are shown in Figures 4-34 to 4-36. Figure 4-34 shows the strain distribution in Girder 1 on Run 1 being very close to linear, and Figure 4-36 show the strain distribution in Girder 3 on all critical runs being close to linear.

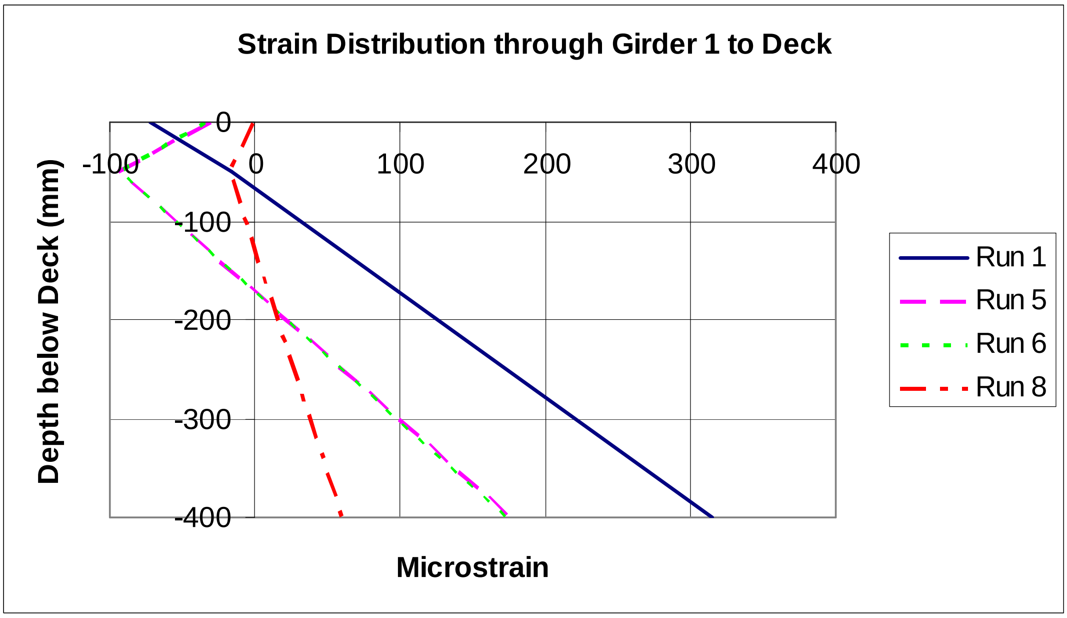

Figure 4-34: Strain through Girder 1 to Deck

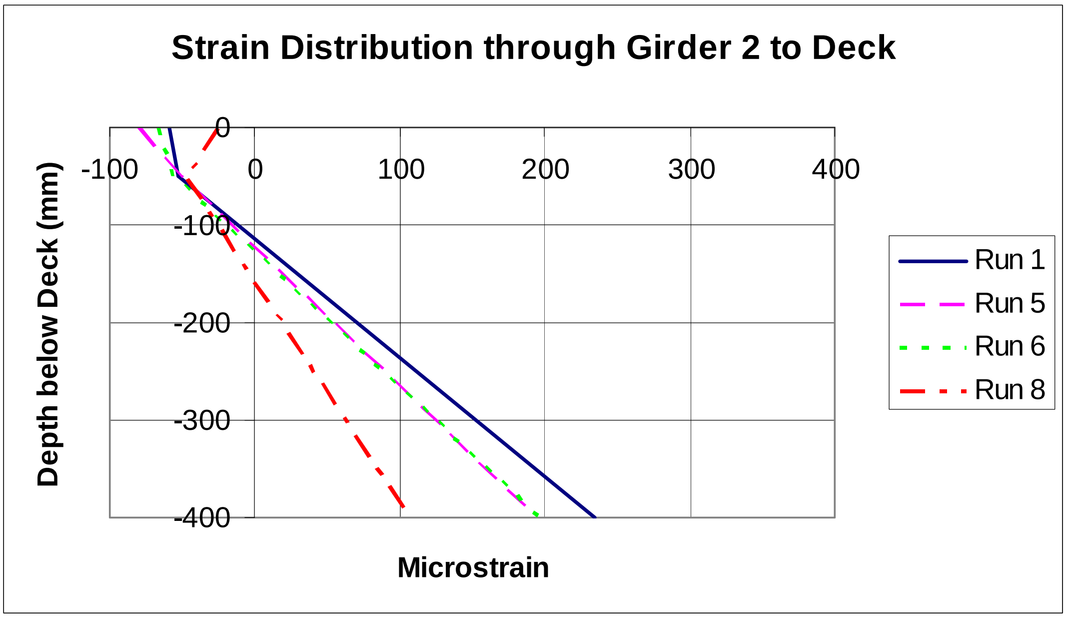

Figure 4-35: Strain through Girder 2 to Deck

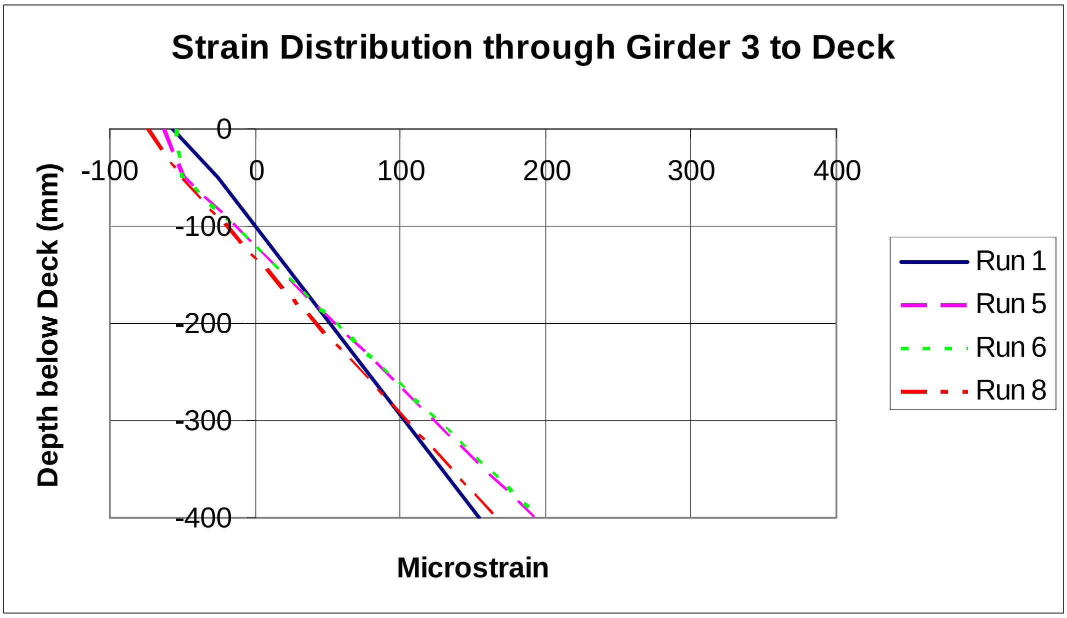

Figure 4-36: Strain through Girder 3 to Deck

4.4.1.5. Shear Strains

The two delta rosette configurations at either end of Girder 1 gave fairly similar shear strain value when using equation 1. Table 4-8 shows the shear strain values calculated from the recorded data and Figures 4-37 to 4-41 show the strain values measured at each end of the bridge for Run 1 and strain values measured at the USQ end of the bridge for Runs 5, 6 and 8. Other strain plots for shear strain calculation can be found in Appendix D, together with the data used to develop the plots.

Table 4-8: Maximum Delta Rosette Measurements and Calculated Shear Strain Values (Critical Runs)

Run #	End	ε (-45°) (μ)	ε (0°) (μ)	ε (+45°) (μ)	γ_{xy} (μ)
1	USQ	211	0	-214	-425
	Handley St	203	-44	-203	-406
5	USQ	167	0	-161	-328
	Handley St	162	-42	-157	-319
6	USQ	175	0	-172	-347
	Handley St	178	-47	-172	-350
8	USQ	127	10	-110	-237
	Handley St	121	-27	-111	-232

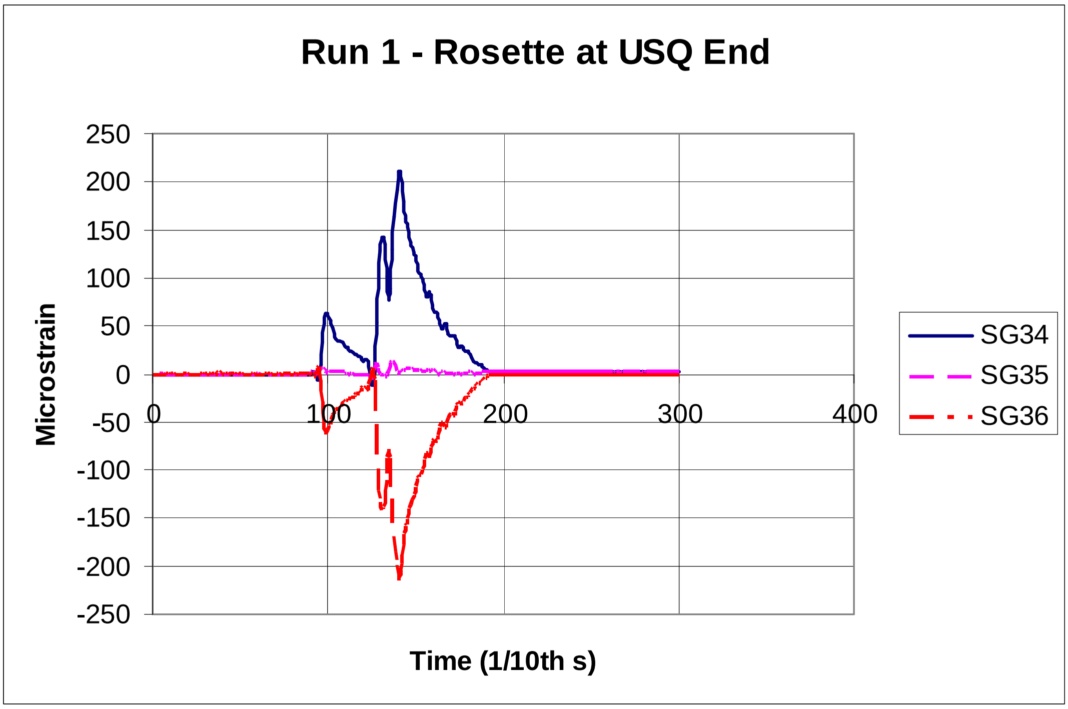

Figure 4-37: Delta Rosette Strains - Run 1 at USQ End

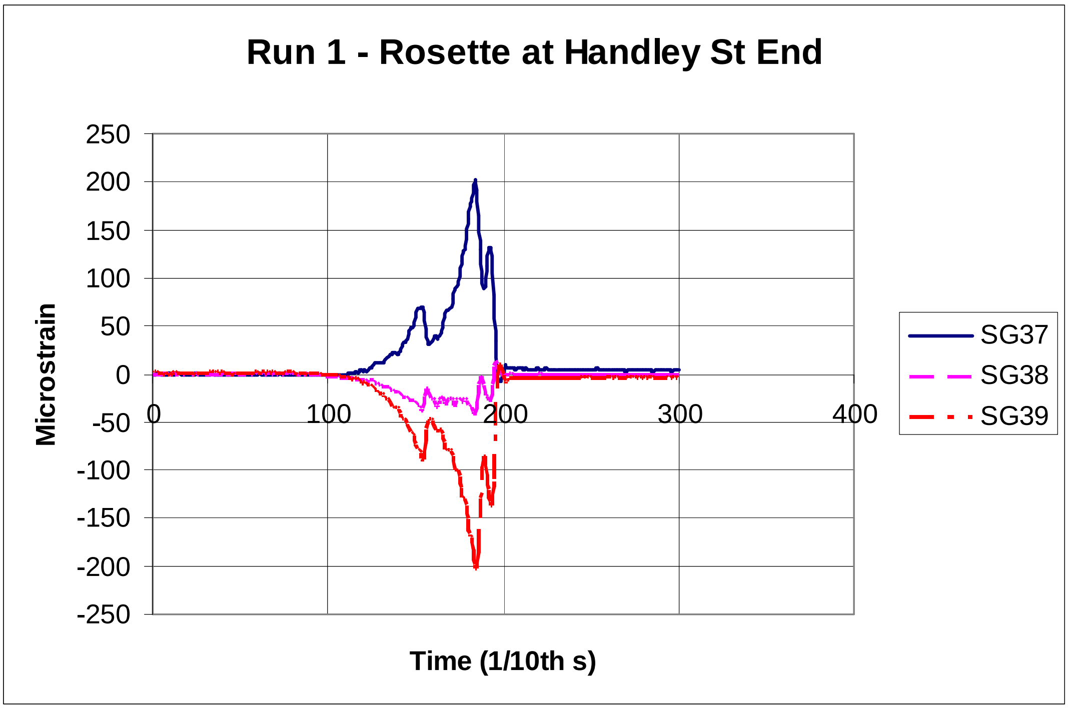

Figure 4-38: Delta Rosette Strains – Run 1 at Handley St End

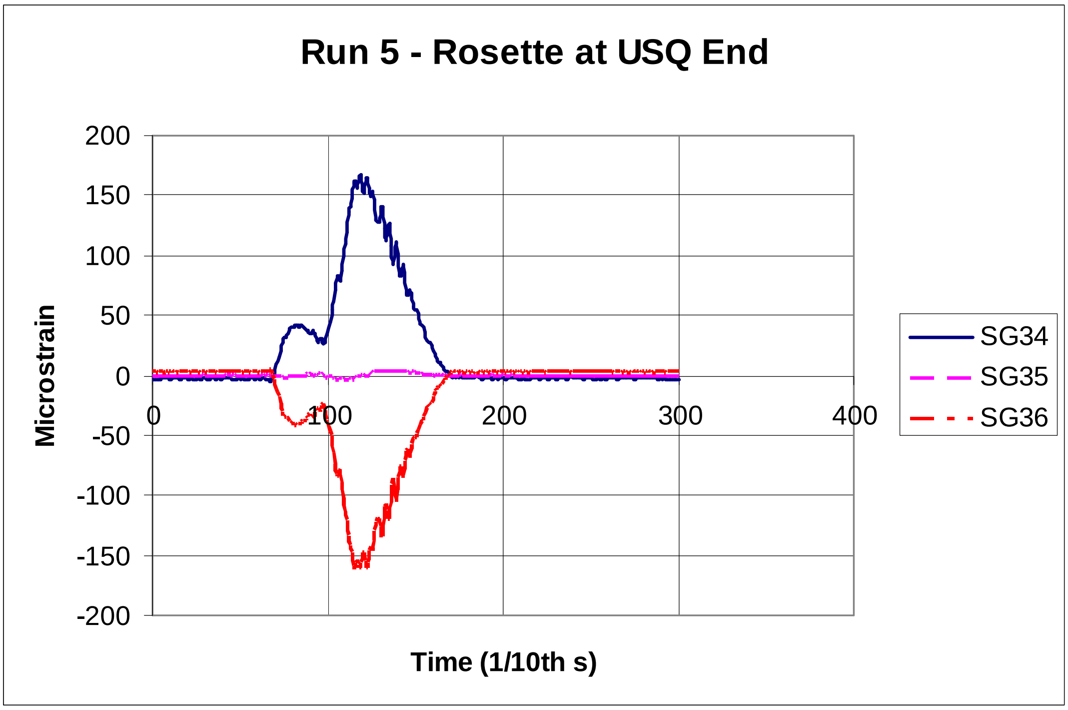

Figure 4-39: Delta Rosette Strains - Run 5 at USQ End

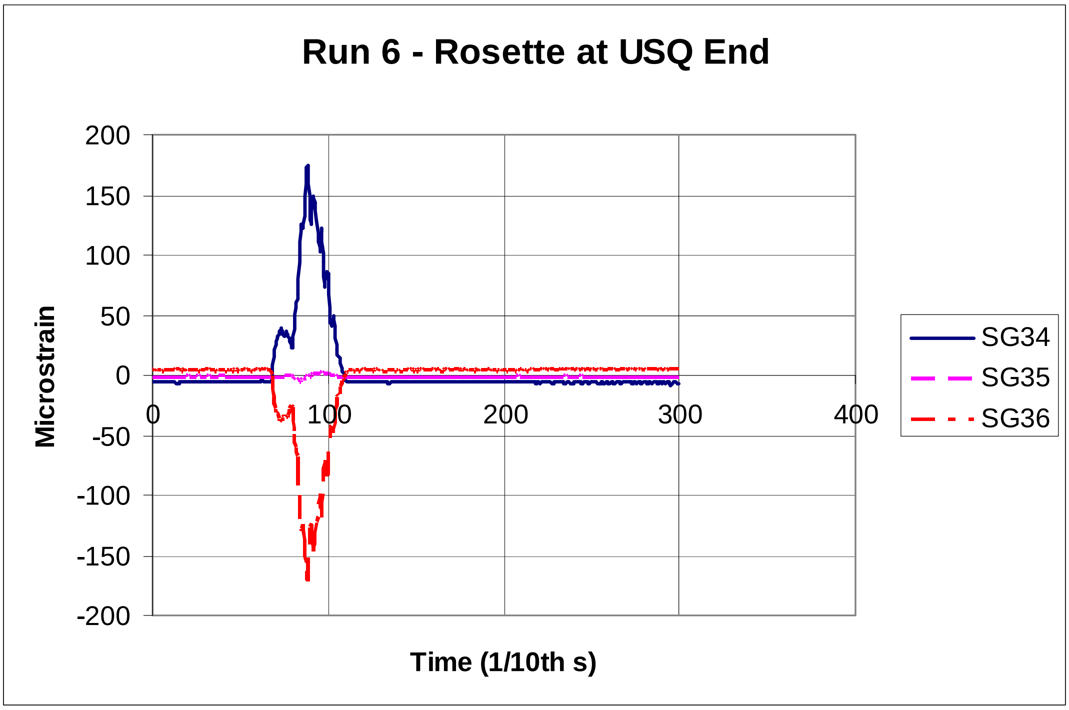

Figure 4-40: Delta Rosette Strains- Run 6 at USQ End

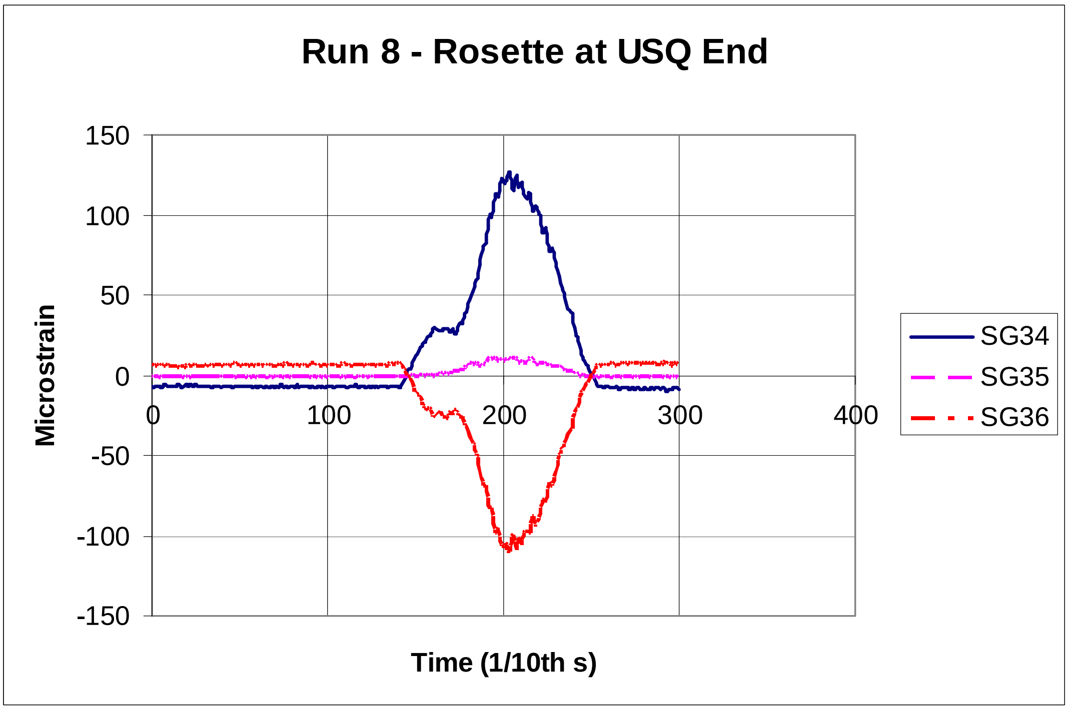

Figure 4-41: Delta Rosette Strains - Run 8 at USQ End

4.4.2. Static Loading Strains

As with static load deflections, the static load strains have been analysed for use in comparison with the dynamic loading strains. This will assist in determining the extent of the dynamic response of the bridge to the truck.

4.4.2.1. Girder Tensile Strains

The maximum tensile strain for each of the girders was between 150 μ and 200 μ (Figure 4-42) under static loading. This compares favourably with maximum tensile strain values of between 170 μ and 220 μ for all girders on Runs 5 and 6 (Figures 4-22 to 4-24 - dynamic loading, central runs).

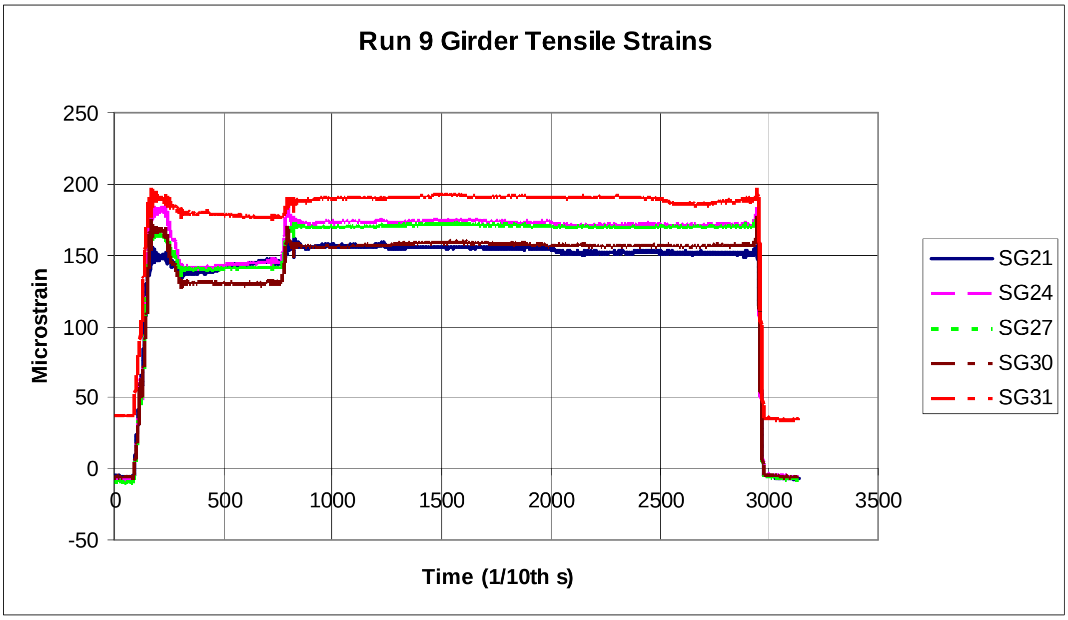

Figure 4-42: Static Loading Girder Tensile Strains

4.4.2.2. Girder Compressive Strains

The maximum compressive strain for each of girders 1 to 3 was between 40 μ and 100 μ (Figure 4-43) under static loading. This compares very favourably with maximum compressive strain values of between 40 μ and 100 μ for girders 1, 2 and 3 on Runs 5 and 6 (Figures 4-26 to 4-28 – dynamic loading, central runs).

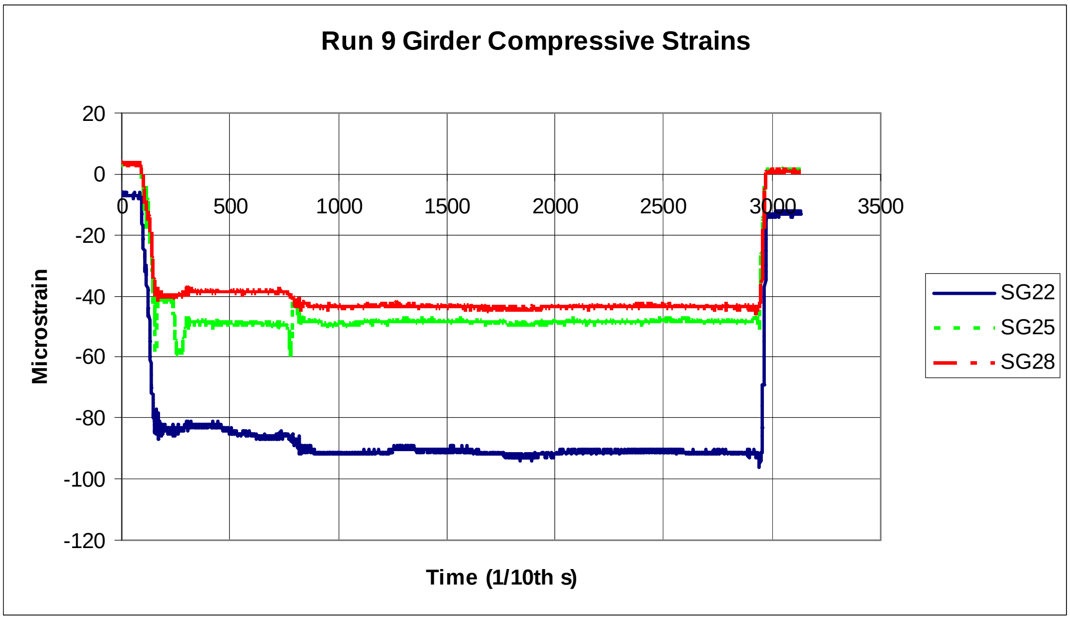

Figure 4-43: Static Loading Girder Compressive Strains

4.4.2.3. Deck Strains

As with the dynamic loading results, both longitudinal and transverse strain have been recorded and analysed.

Longitudinal Deck Strain

The maximum compressive longitudinal deck strain for each of the deck panels near girders 1, 2 and 3 was between 20 μ and 80 μ (Figure 4-44) under static loading. This compares favourably with maximum compressive strain values of between 30 μ and 100 μ for recordings taken on Runs 5 and 6 (Figures 4-29 to 4-31 – dynamic loading, central runs).

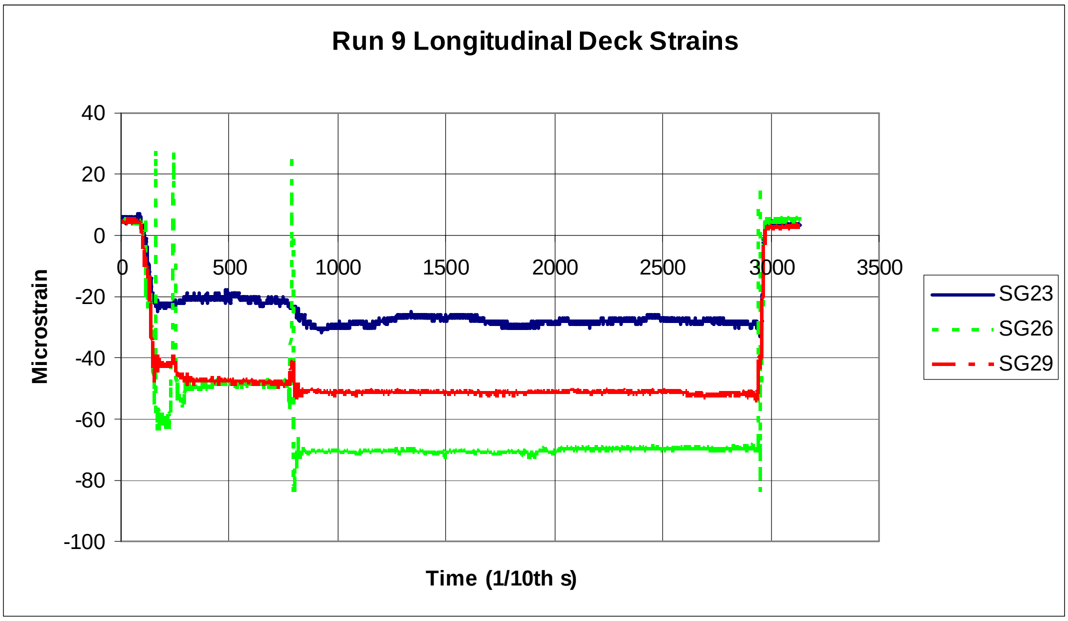

Figure 4-44: Static Loading Longitudinal Deck Strains

Transverse Deck Strains

The maximum tensile strain of 30 μ to 40 μ between girders 1 and 2, and maximum tensile strain of 250 μ to 260 μ between girders 2 and 3 (Figure 4-45 – static loading) compares favourably with those of 40 μ to 60 μ and 240 μ to 280 μ (Figures 4-32 and 4-33 – dynamic loading, central runs).

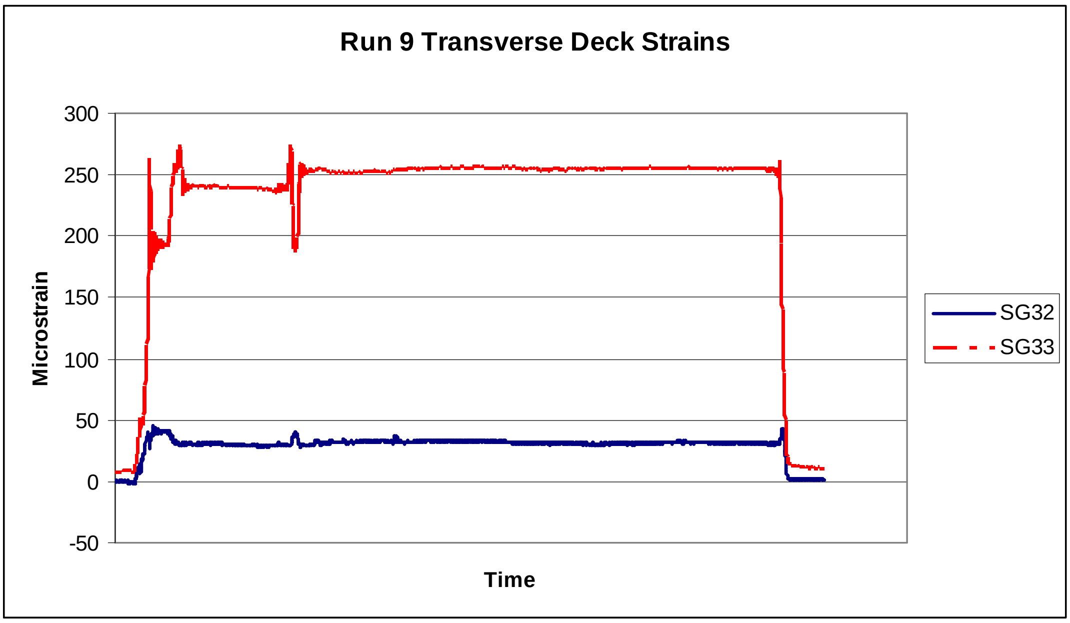

Figure 4-45; Static Loading Transverse Deck Strains

4.4.2.4. Combination Strains

The combination strain through girders 1, 2 and 3 under static loading shows a very close relationship with the dynamic loading combination strains on the central runs (Runs 5 and 6). This can be seen in Figures 4-46 to 4-48.

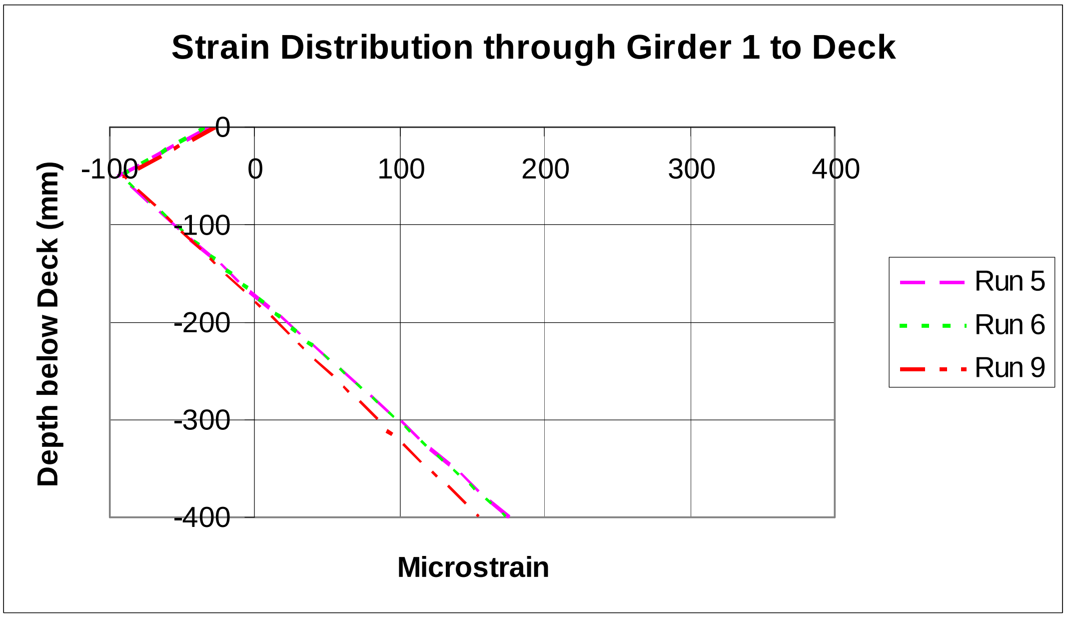

Figure 4-46: Strain through Girder 1 to Deck

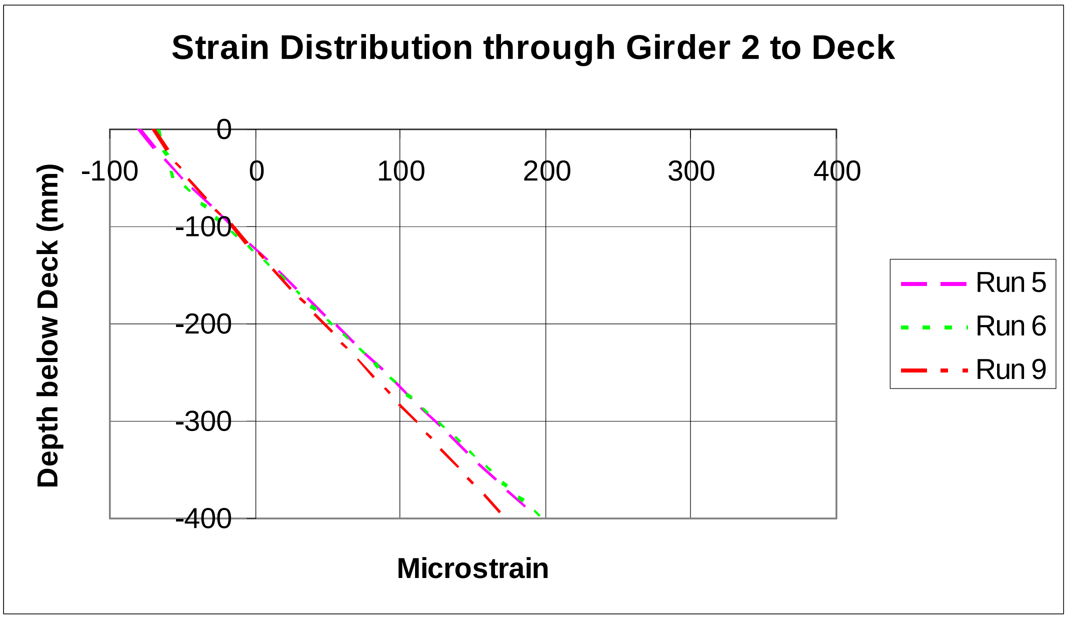

Figure 4-47: Strain through Girder 2 to Deck

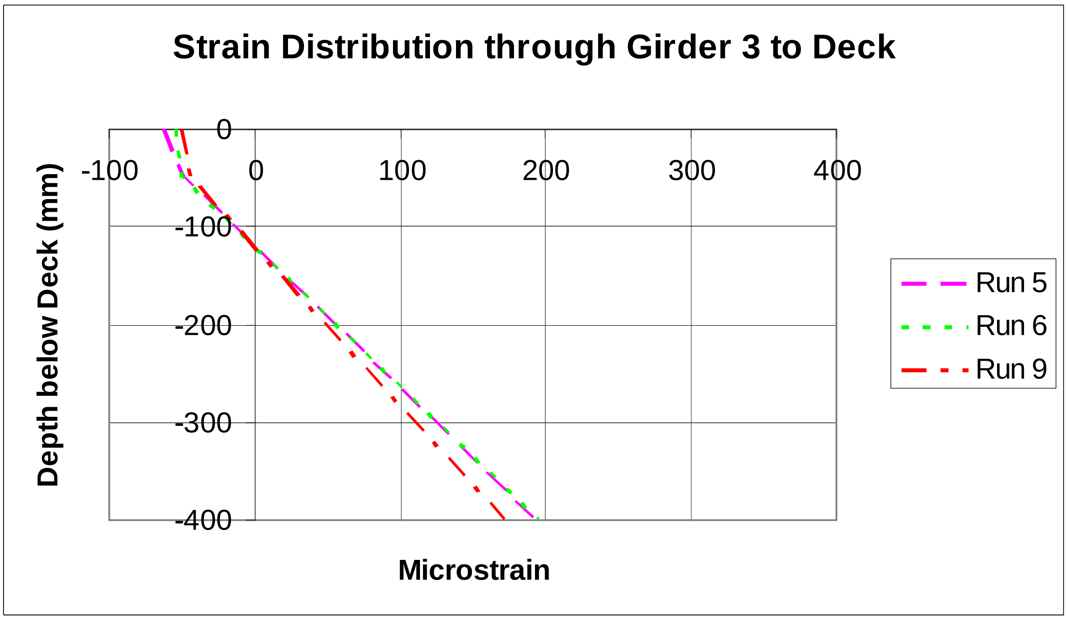

Figure 4-48: Strain through Girder 3 to Deck

4.4.2.5. Shear Strains

The comparison between the shear strains on dynamic loading runs (Runs 5 and 6) and the shear strains on the static loading run (Run 9) are quite different to the other strain comparisons. As calculated previously, the maximum shear strains on the dynamic loading runs were between -319 μ and -350 μ, whereas the calculated maximum shear strains (Figures 4-49 and 4-50) on the static loading run ranged between -214 μ and -222 μ (Table 4-9).

Table 4-9: Maximum Delta Rosette Measurements and Calculated Shear Strain Values (Dynamic and Static Central Runs)

Run #	End	ε (-45°) (μ)	ε (0°) (μ)	ε (+45°) (μ)	γ_{xy} (μ)
5	USQ	167	0	-161	-328
	Handley St	162	-42	-157	-319
6	USQ	175	0	-172	-347
	Handley St	178	-47	-172	-350
9	USQ	109	0	-105	-214
	Handley St	113	-28	-109	-222

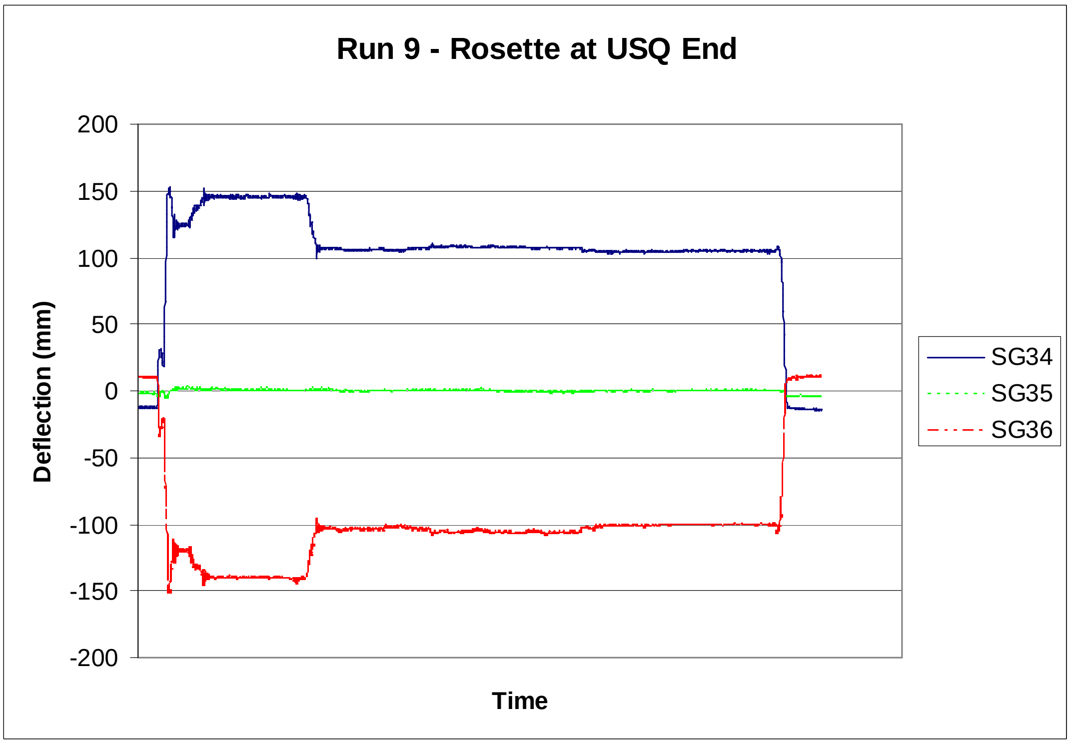

Figure 4-49: Delta Rosette Strains – Run 9 at USQ End

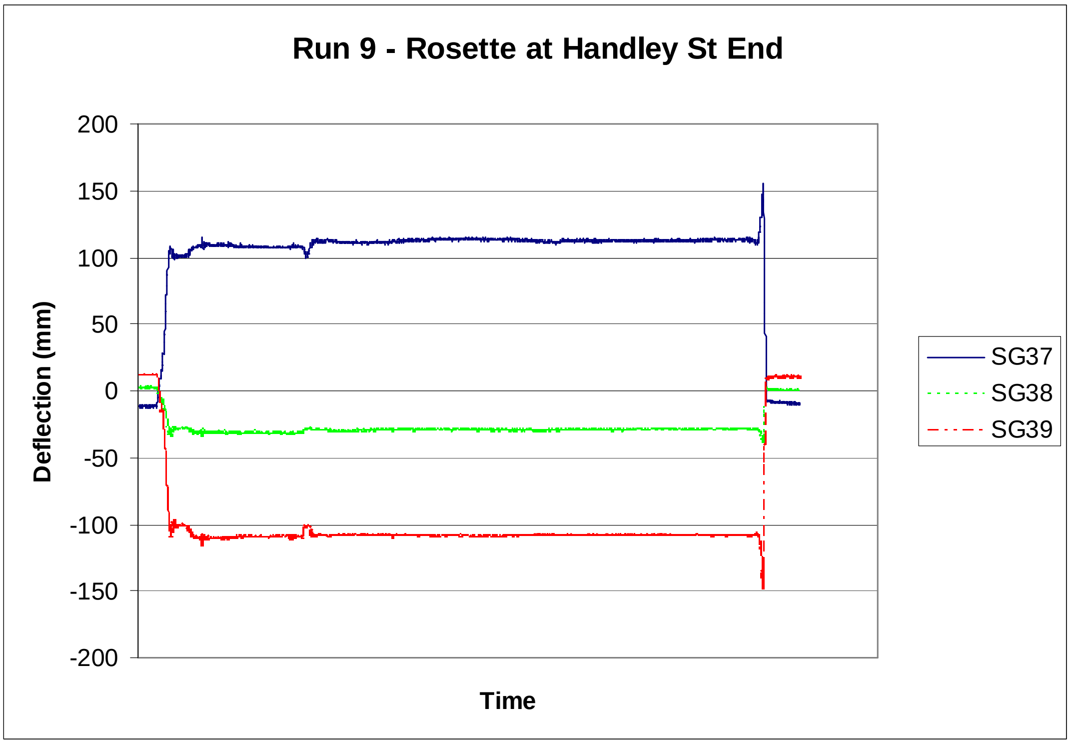

Figure 4-50: Delta Rosette Strains – Run 9 at Handley St End

The static loading shear strains are approximately two-thirds magnitude of the dynamic loading shear strains. This would suggest that there is some induced dynamic shear strain present. As all shear strain values are well below the shear yield capacity of the girders, this appears structurally insignificant, but could be studied further in future projects.

4.5. Summary

The analysis of the field test data showed that mid-span deflection is the governing criteria when designing the bridge, as all of the strain measurements and calculations were much smaller than the yield strains of the members.

The maximum mid-span deflection of the beams is well within the limiting value of 20 mm. As the analyses were conducted with the data from phase 2 of the loading (overloaded truck), this suggests a significant factor of safety is present if this bridge were to be installed into the Queensland road network.

The magnitude of the difference in dynamic load deflections and static load deflections was quite small (< 1 mm), and would appear to be structurally insignificant; however, this could be an area of further study in the future.

The magnitude of both girder-deck differential deflections and deck panel differential deflections would also appear to have no structural significance (0.2 to 0.5 mm). Again, this could be an area where future study may take place.

On the runs where the wheels travelled directly over the girders being analysed, the load comparison analysis and the strain distribution analysis exhibited reasonably consistent linearity of results. This would tend to suggest that there may be some out of plane effects occurring in the other girders that may skew the results. The magnitude of any skewing does not appear to have great structural significance, but may be an area of future research.

Both ends of girder 1 showed similar calculated shear strain values. Assuming that all girders were produced in an identical fashion, this analysis suggests that the cross section of the girders is fairly consistent.

The magnitude of the shear strain induced by the dynamic loading was greater than that induced by the static loading. This may be caused by some bouncing of the truck as it crosses uneven deck joints.

Time constraints did not allow for the inclusion of natural frequency testing data analysis. The data collected from the natural frequency field testing should be available for analysis at a later date, if required.

All field data collected and analysed can be found in Appendix D.

5. FINITE ELEMENT ANALYSIS

5.1. Introduction

The Strand7 software package was chosen for use as the finite element analysis tool in this project. A simple grillage model was developed, analysed and then modified and re-analysed to obtain maximum mid-span deflections for use in comparison with field data analysis.

5.2. Development of Grillage Model

After becoming familiar with the working of the Strand7 Finite Element Analysis software package, the model of the bridge was developed.

5.2.1. Girder Design

The girders were designed as 400 mm by 400 mm square beams, with a previously determined EI value of 5.66×10^{13} Nmm^2 (calculated from prior testing by the CEEFC). This gave a design E (modulus of elasticity) value of 26531 MPa. A Poisson's ratio value of 0.3 was used for the girder design. The girders were subdivided into 500 mm equal sections and connected with transverse diaphragms at the appropriate points.

5.2.2. Diaphragm Design

The diaphragms were designed in similar fashion to the girders, with the dimensions being 250 mm by 400 mm rectangular beams. Due to the lack of information available, the E value of the girders was used for the diaphragm design, and a Poisson's ratio value of 0.3 was also used.

5.2.3. Freedom Conditions and End Restraints

For the preliminary model, one end of the bridge was designed as a simple support (only allowed movement is rotation about the z-axis) and the other end of the bridge was designed as a roller support (allowed movements are translation in the x direction and rotation about the z-axis).

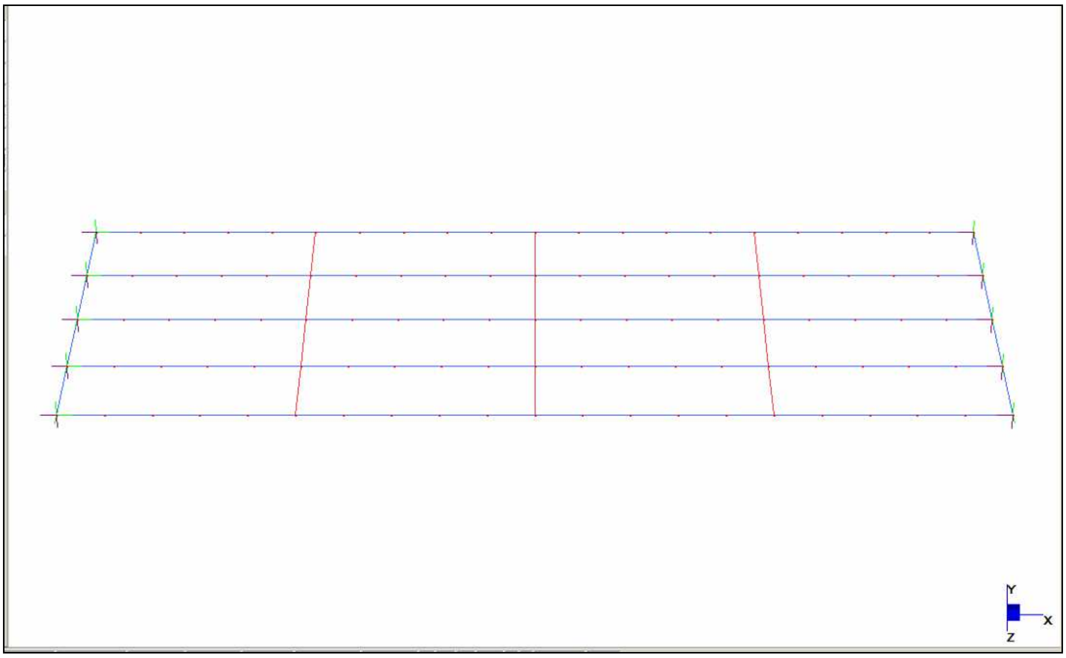

Figure 5-1: Initial Strand7 Model

5.2.4. Loading

The truck axle loads were used as wheel point loads on girders 2 and 4 for centre run analysis, and girders 3 and 5 for side run analysis, as shown in Figures 5-2 and 5-3. The point loads were moved from right to left along the bridge in 1 m increments for each load case (17 load cases for each model), starting at the right hand end of the model (load case 1).

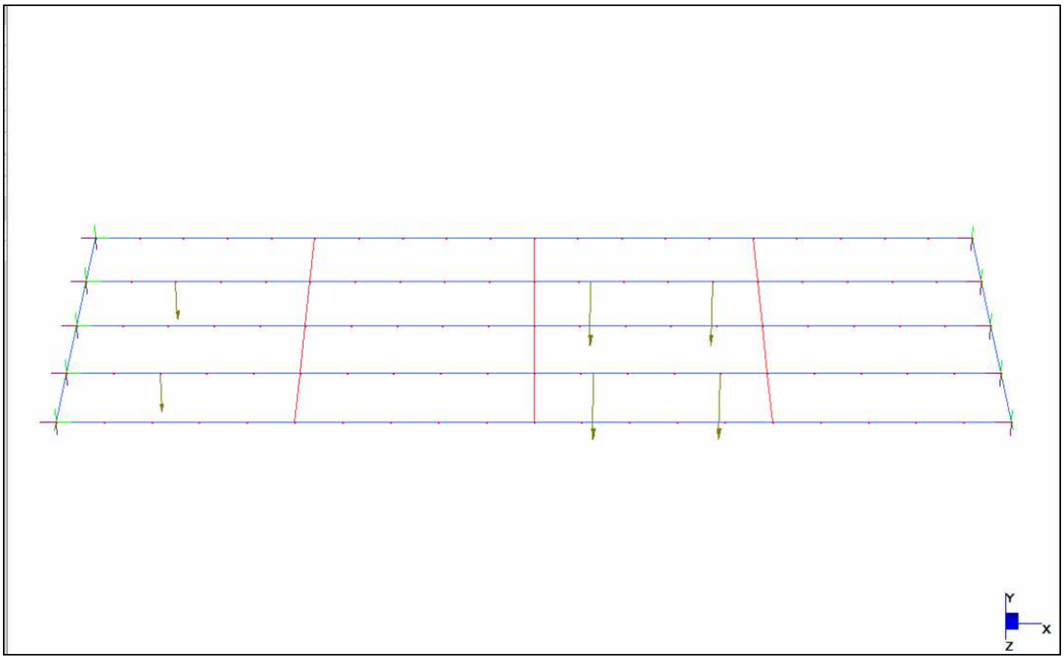

Figure 5-2: Addition of Loading for Central Runs

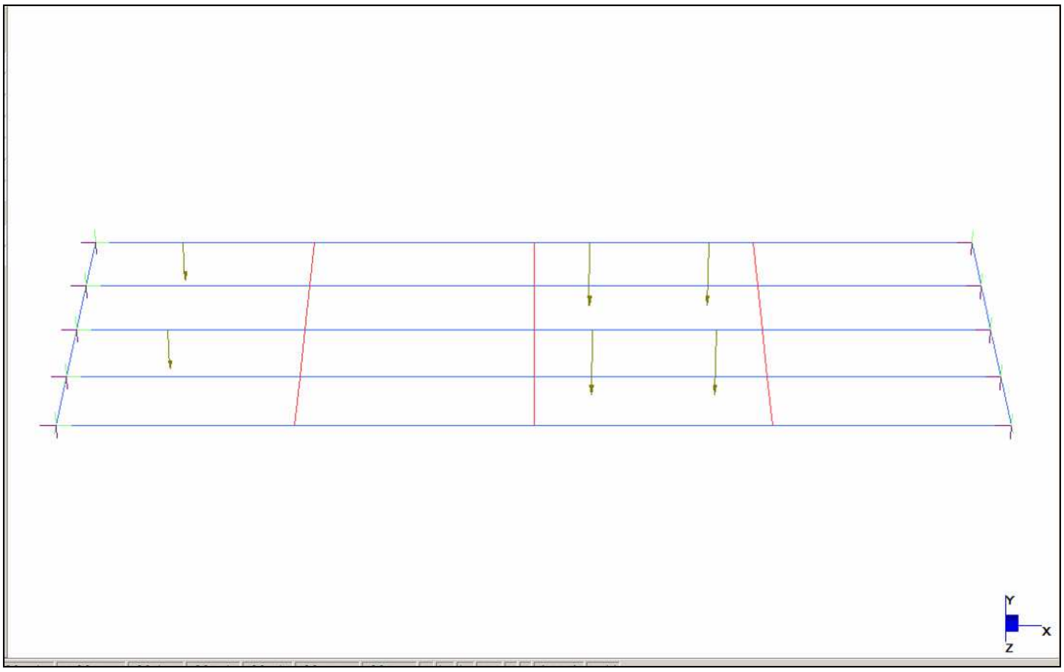

Figure 5-3: Addition of Loading for Side Runs

5.3. Analysis of Grillage Model

5.3.1. Centre Run and Side Run Analysis

Both centre run and side run models were analysed using a linear static analysis and results collated in Table 5-1. The maximum deflections were created under load case 10 and load case 11 conditions (rear axles had not crossed mid-span). The maximum deflections of both models (load case 10 for centre run, load case 11 for side run) are shown in Figures 5-4 and 5-5.

Table 5-1: Strand7 FEA Grillage Model Results

Girder #	Run Type	Maximum Mid-span Deflection (mm)
1	Centre	14.9
	Side	17.7
2	Centre	15.3
	Side	16.4
3	Centre	15.4
	Side	15.2
4	Centre	15.3
	Side	13.9
5	Centre	14.9
	Side	12.7

From Table 5-1, it can be seen that all maximum mid-span deflections are less than the maximum allowable deflection of 20 mm (span/500).

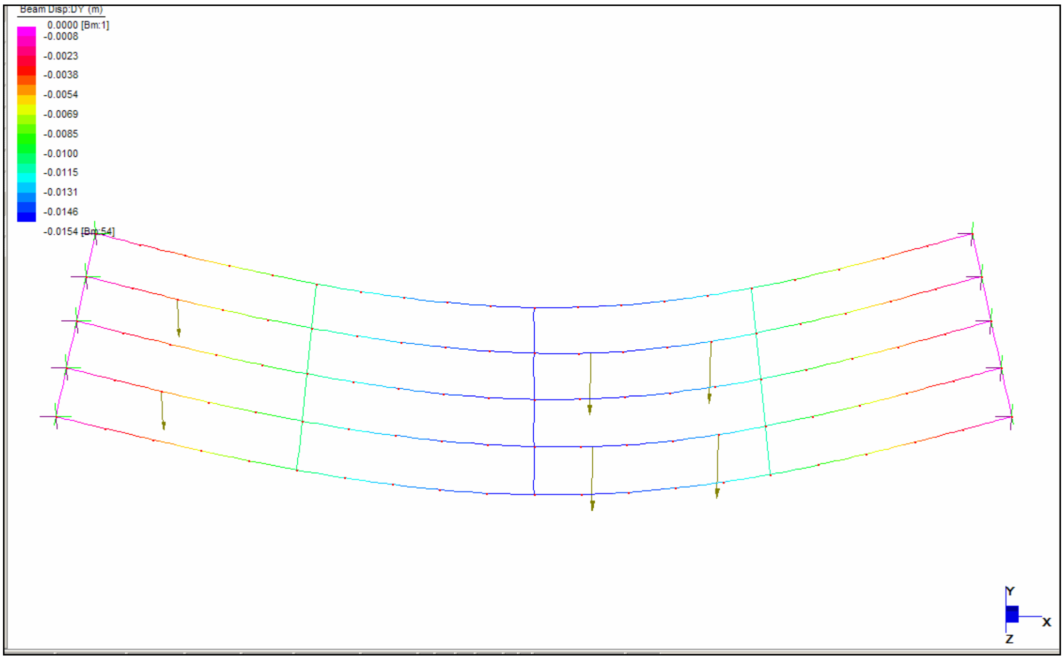

Figure 5-4: Maximum Deflection of Grillage Model - Centre Run

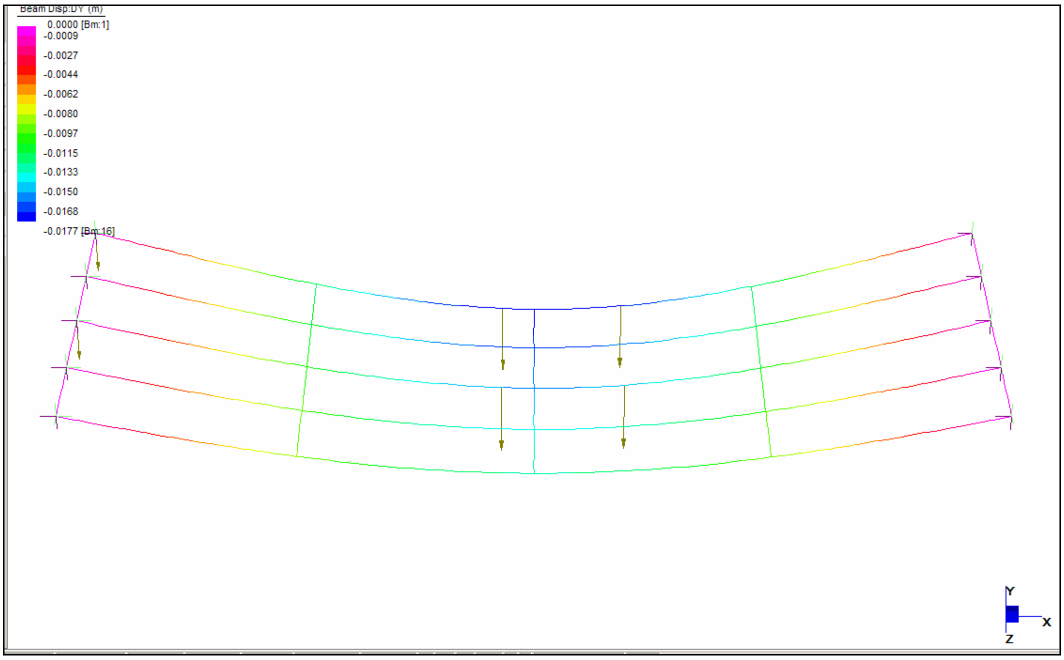

Figure 5-5: Maximum Deflection of Grillage Model - Side Run

Figures 5-6 and 5-7 show the predicted deflections of the girders from the Strand7 grillage model analysis.

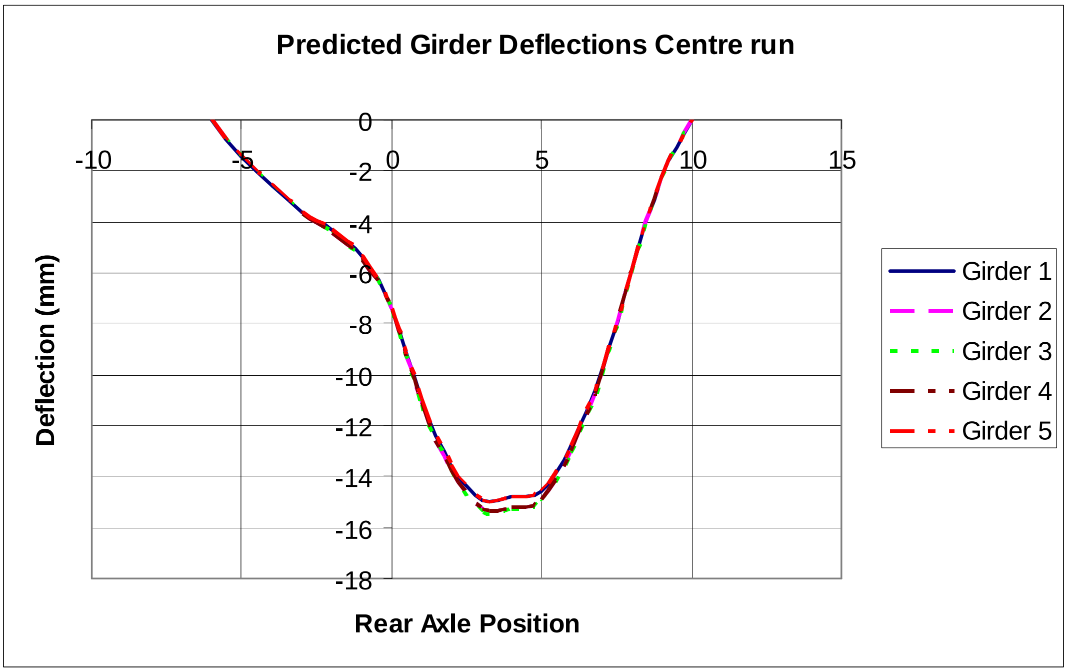

Figure 5-6: Predicted Girder Deflections - Centre Run

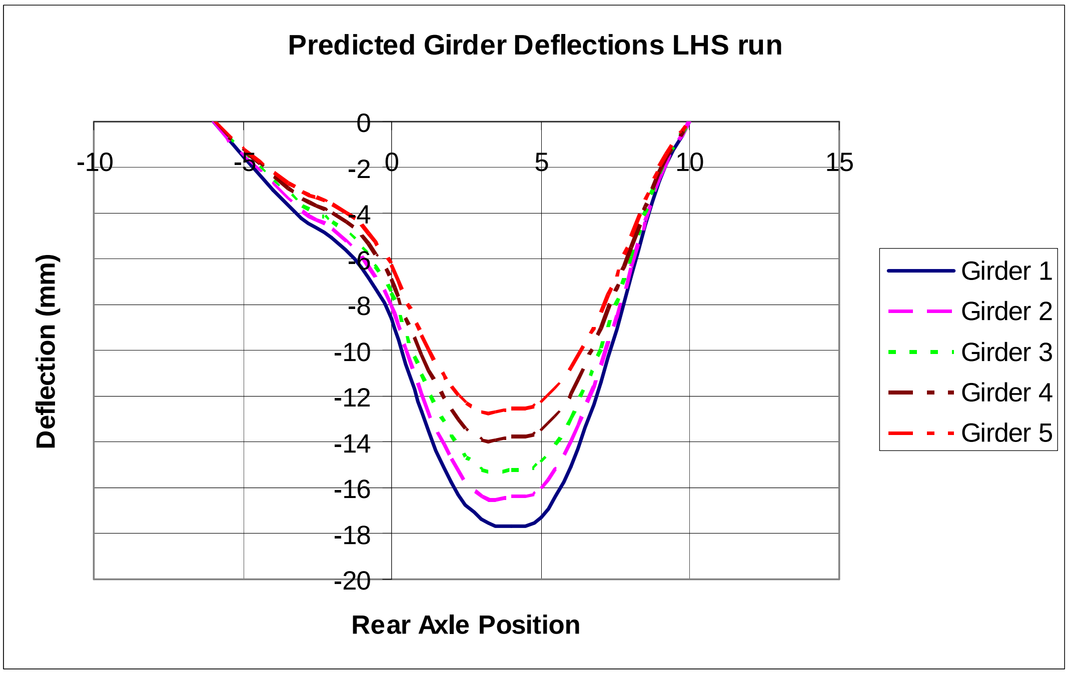

Figure 5-7: Predicted Girder Deflections - Side Run

5.3.2. Diaphragm Modification

The widths of the diaphragms were modified to investigate whether reducing the size of the diaphragms significantly altered the maximum deflections created by the Strand7 analysis. As Table 5-2 shows, the reduction of diaphragm width only increases the maximum deflection marginally; however, removing the diaphragms altogether increases the maximum deflections above the limiting value of 20 mm.

Table 5-2: Effect of Diaphragm Modification on Maximum Deflections - Strand7 FEA Grillage Model

Girder #	Run Type	Diaphragm Width (mm)	Maximum Deflection (mm)
1	Side	250	17.7
		150	17.9
		50	18.1
		-	24.6
		400	17.5
3	Centre	250	15.4
		150	15.4
		50	15.5
		-	21.4
		400	15.3

5.4. Addition of Deck to Grillage Model

Two approaches to addition of the deck were considered. In both approaches, analysis was undertaken using 250 mm wide diaphragms. The entire deck consisted of 320 plate elements.

5.4.1. Deck Modelled as Laminate (Approach 1)

Initially no previous testing was considered, and the deck was considered as a laminate, with five Triax/core layers and a 25 mm thick concrete layer at the top. Each Triax/core layer consisted of three Triax UD 250 gsm sheets, oriented at +45°, 0° and -45° to horizontal, covering a central core, with three more Triax sheets on the other side, oriented at -45°, 0° and +45° to horizontal. This is shown in Figure 5-8.

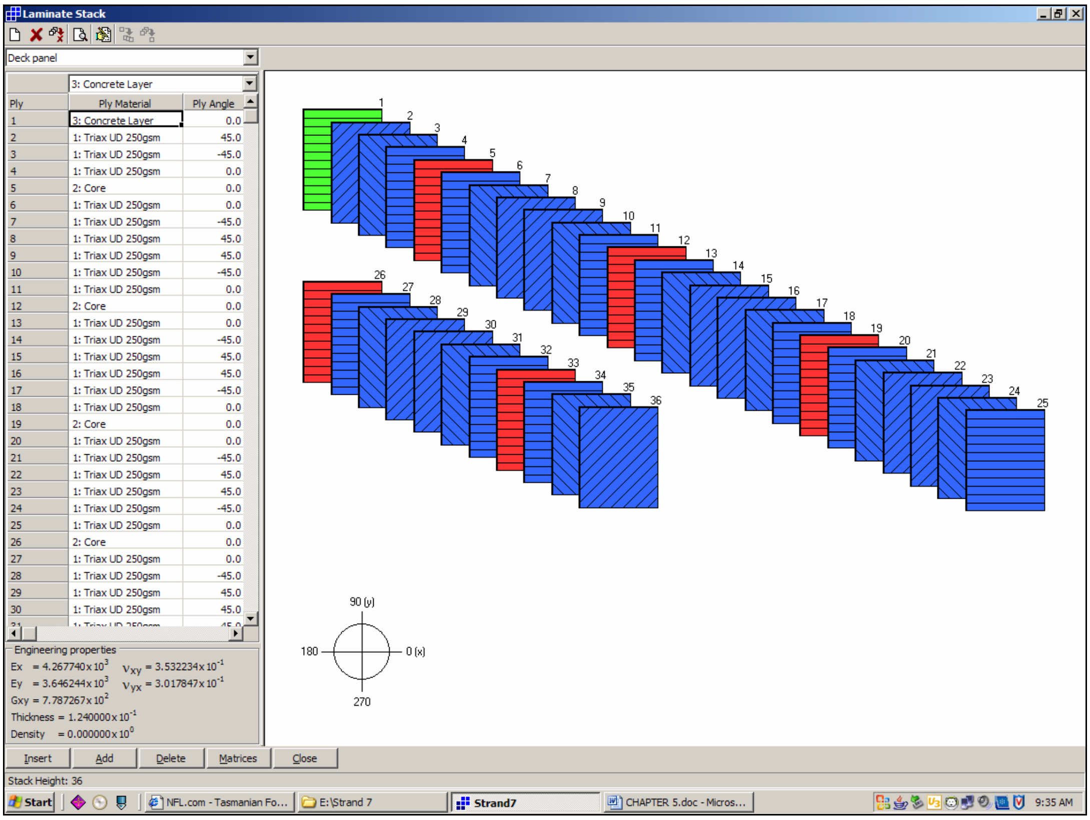

Figure 5-8: Configuration of Deck Panels for FEA

This configuration gave approximate E_x and E_y values of 4267 MPa and 3646 MPa respectively. Using the properties of this configuration for the plate elements, linear static analysis was undertaken and the deflections noted.

5.4.2. Deck Modelled using Experimental Data (Approach 2)

Using the second approach, previous testing (three point loading, November 2005) of 390 mm wide, 1100 mm long and 120 mm thick deck panels had been performed, and from the load-deflection plot an approximate E value of 3300 MPa was calculated. Due to confidentiality issues, no prior test results can be shown in this project.

5.5. Finite Element Analysis with Deck Added

The results of the finite element analysis using the first deck model are shown in Table 5-3. As the results show, the addition of the deck to the model only reduces the maximum mid-span deflection by 0.4 mm (Approach 1) and 0.1 mm (Approach 2). It did not matter which approach to the modelling of the deck was used. The magnitude of this reduction is structurally insignificant, and would therefore be neglected. A simple grillage model would then become the preferred model as it is quicker to develop whilst giving almost the same results as the model with the deck included. The simple grillage model will also tend to give slightly more conservative results, so can be used with greater confidence.

Table 5-3: Strand7 FEA results with Deck added to Model

Girder #	Run Type	Deck added (Yes/No)	Deck Approach Used (1/2)	Maximum mid-span deflection (mm)
1	Side	No		17.7
		Yes	1	17.3
		Yes	2	17.6
3	Centre	No		15.4
		Yes	1	15.0
		Yes	2	15.3

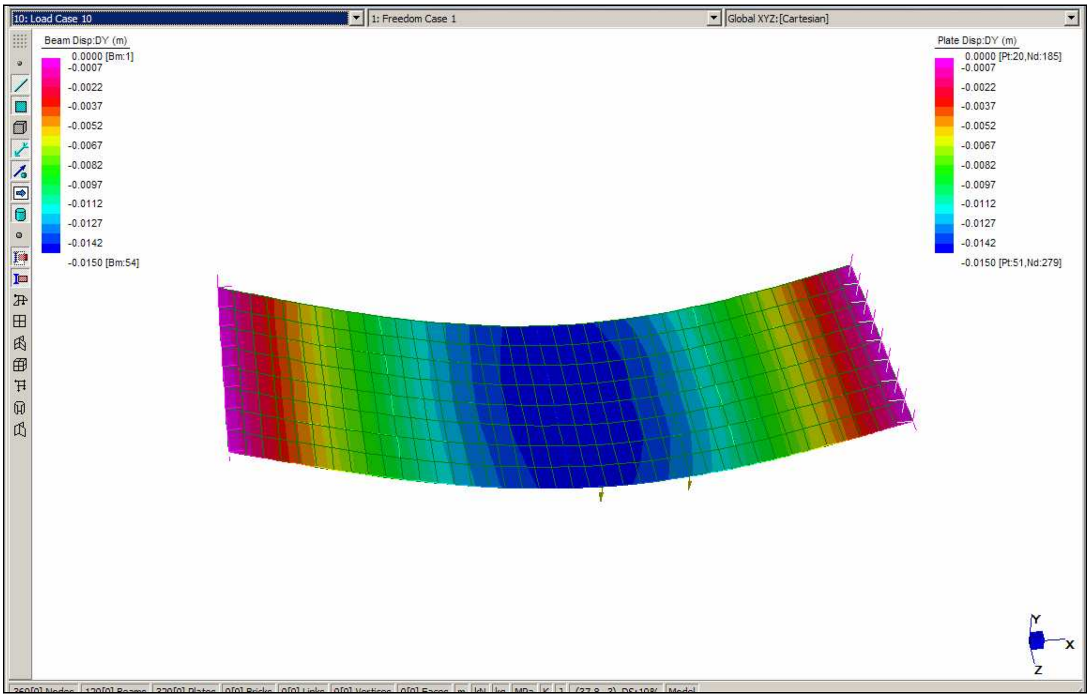

Figure 5-9: Maximum Deflection of Grillage Model with Deck 1 added - Centre Run

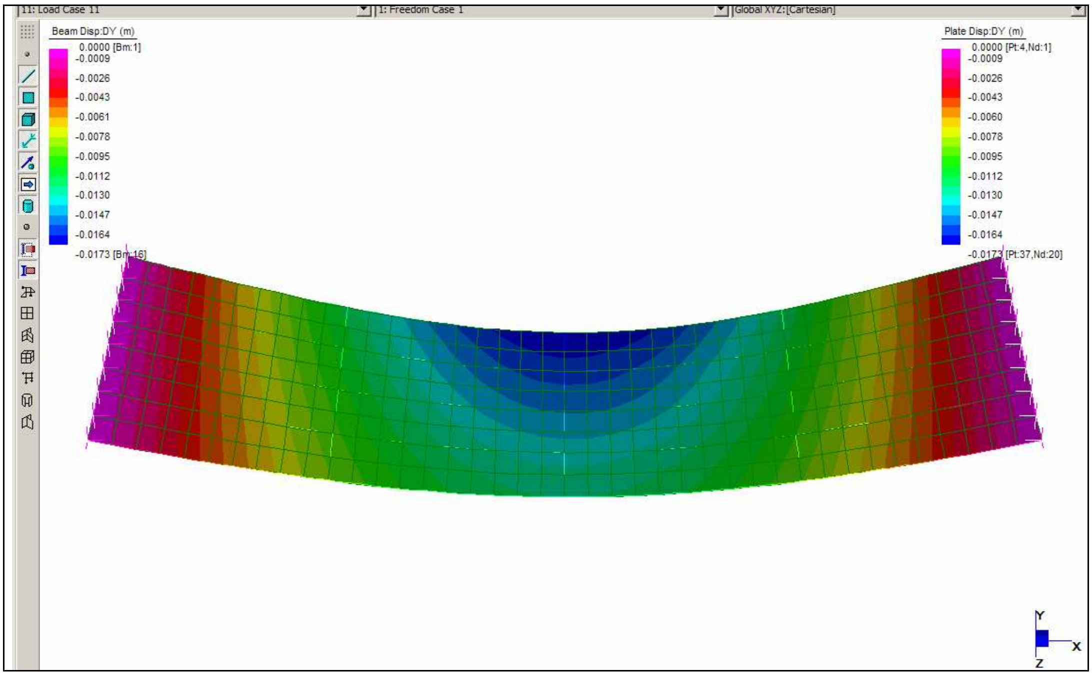

Figure 5-10: Maximum Deflection of Grillage Model with Deck 1 added - Side Run

5.6. Extension of Bridge to Full-size

The model was developed assuming that the dimensions of the girders, diaphragms and deck panels did not change, and that the distances between girders and diaphragms remained constant. Under simple loading it could be expected that the maximum mid-span deflection would increase by a factor of 8 due to the deflection being proportional to the length cubed i.e. if the length is doubled, the deflection increases eightfold. The maximum mid-span deflection predicted using the simple grillage model would be approximately 125 mm for the centre run, and 142 mm for the side run.

5.6.1. Development of Model

As with the initial model, the girders were created first followed by the diaphragms. The girders were subdivided into 500 mm equal sections and connected with transverse diaphragms at the appropriate points. The load cases were set up in the same way as for the first model (1 m increments from right to left across the bridge).

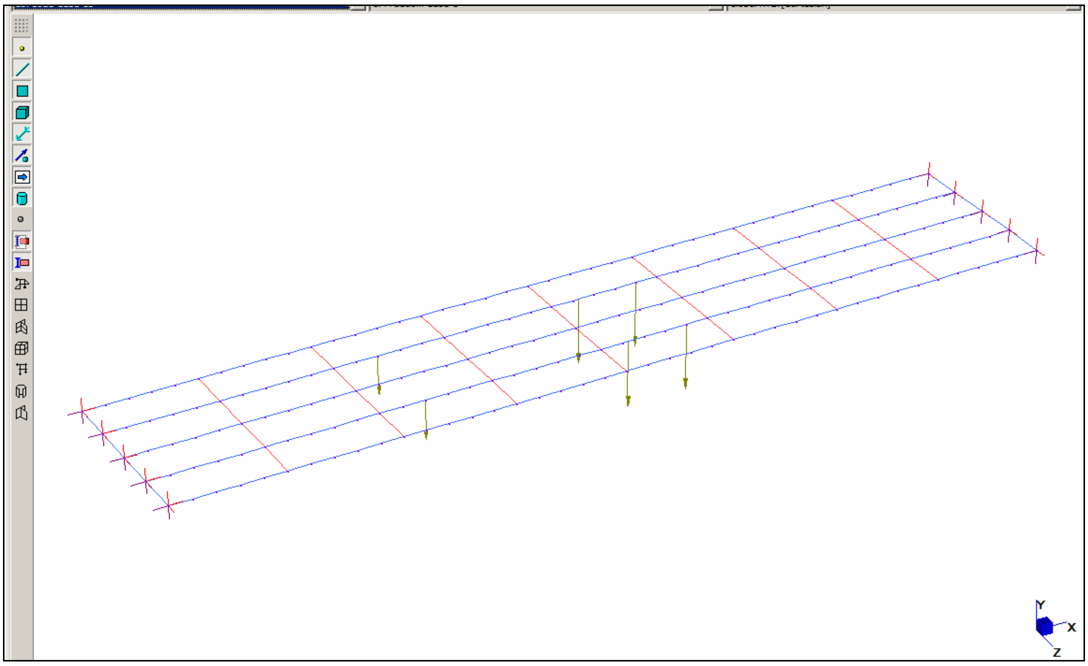

Figure 5-11: Full-size Strand7 Model

5.6.2. Full-size Model Analysis

The analysis of the full-size model gave a predicted deflection of just over 150 mm for the centre run and approximately 158 mm for the side run. This is somewhat higher than expected, particularly for the centre run, and has not been researched further. Investigating these results and the issues arising from them could be another area for future study.

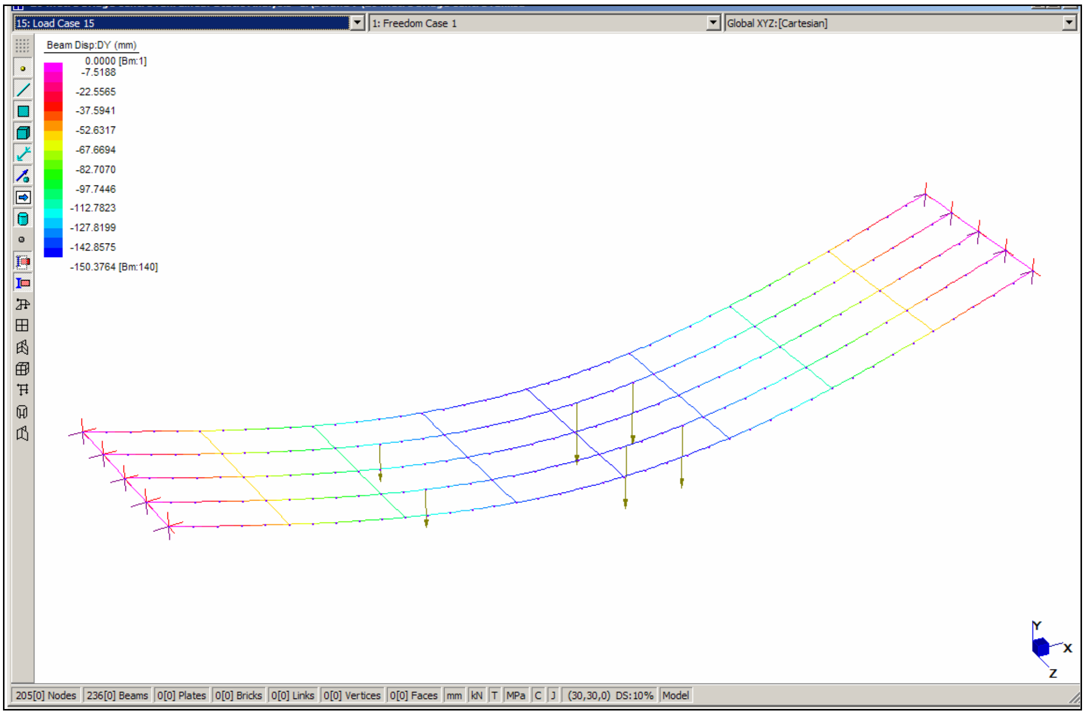

Figure 5-12: Maximum Deflection of Full-size Grillage Model - Centre Run

Table 5-4: Predicted and FEA Deflections for Full-size Bridge

Girder #	Run Type	Approx. Predicted Deflection (mm)	Approx. FEA Deflection (mm)
1	Side	125	150
3	Centre	142	158

5.7. Summary

From the results of the various finite element analyses, it can be seen that a simple grillage analysis can be undertaken to predict deflections, as the addition of the deck elements did not affect the results enough to warrant taking the extra time to put the deck into the model.

When considering the predicted deflections from the finite element analysis, it must be understood that finite element analysis tends to give fairly conservative results. Care must also be taken to use correct material properties for all of the elements, particularly dimensions and E values. As there were only approximate values available for the finite element analysis undertaken in this project, the results obtained should be used with some caution.

When undertaking the finite element analysis of the full-size bridge model, a simple grillage model could be used. Modification of the various beam and diaphragm parameters (dimensions, E values) and of the end restraint conditions could be employed to allow the predicted maximum mid-span deflections to become less than the limiting deflection of span/500, in this case 40 mm. These parameter values could then possibly be used in the design and construction of the full-size bridge.

6. ANALYSIS COMPARISONS

6.1. Introduction

The comparison of field testing analysis results with finite element analysis (FEA) results can be useful when deciding whether to use finite element analysis tools in predicting deflections in prospective construction projects. Given that historical finite element analysis has tended to be conservative, it can be said with some degree of certainty that if predicted finite element analysis results are within acceptable limits, then construction should be able to be undertaken safely.

6.2. Deflections

6.2.1. Mid-span Deflections

From the field results, the most important values to consider when comparing with finite element results are the maximum deflections of girders 1 and 5 (outside girders) and girder 3 (central girder). The maximum mid-span deflections of all girders are shown in Table 6-1.

Table 6-1: Maximum mid-span Deflections (Field Testing)

Girder #	Maximum mid-span deflection (mm)
1	12.633
2	10.279
3	8.434
4	8.968
5	11.505

The maximum mid-span deflections from the finite element analysis of the simple grillage model with 250 mm wide diaphragms are shown in Table 6-2 (using girder 1 results for girders 1 and 5, and girder 2 results for girders 2 and 4).

Table 6-2: Maximum mid-span Deflections (FEA)

Girder #	Maximum mid-span deflection (mm)
1	17.7
2	16.4
3	15.4
4	16.4
5	17.7

Table 6-3 shows the comparisons and percentage increase in the FEA results compared to the field test results. These results validate the historical conservancy of finite element analysis, but the magnitude of the difference between the two analyses is not easy to justify.

Table 6-3: Comparison of Mid-span Deflections

Girder #	Field Test Deflection (mm)	FEA Deflection (mm)	% Difference
1	12.633	17.7	+ 40.1
2	10.279	16.4	+ 59.5
3	8.434	15.4	+ 82.3
4	8.968	16.4	+ 82.9
5	11.505	17.7	+ 53.8

Possible reasons for the discrepancy may include: error in the instrumentation readings, error in the data analysis, or errors in the finite element model. The most plausible possibility at this point would be errors in the model. The effect of girder-deck interaction is not easy to model in Strand7, so would tend to have the most bearing on the FEA results. This is an area where further study may help in finding the predominant reason for the magnitude of the differences between the field test results and the FEA results.

Non-linear analysis may also have given more realistic predicted deflections; however the magnitude of the difference between linear analysis results and non-linear analysis results was structurally insignificant and non-linear analysis was not considered further.

Changing the restraint conditions of the bridge ends in the FEA model may also have some effect on the mid-span deflections obtained. As the bridge girder ends are bolted and glued (rigid bond) to the abutments, very little or no horizontal movement should occur at the girder ends, whereas having a roller restraint at one end of the model allows for some horizontal movement. Time limitations did not allow this modification of the model to be studied, but should be considered for future projects of this nature.

6.2.2. Deflections near Abutments

As only girder 1 readings were taken from the field testing, a full comparison of results can not be undertaken. Table 6-4 shows the maximum field testing deflection on Runs 1, 5 and 6, the FEA deflection from side and centre runs, and the difference between the two analyses.

Table 6-4: Comparison of Deflection near Abutments - Girder 1

Field Test Run #	Maximum Deflection (mm)	FEA Run Type	Maximum Deflection (mm)	% Difference
1	1.926	Side	2.8	+ 45.4
5	1.048	Centre	2.5	+ 138.5
6	1.008	Centre	2.5	+ 148.0

The only possible reason for the discrepancy in results could be due to the bridge model end restraints allowing rotation in the z-axis, whereas the actual bridge had very little or no rotation at the abutments. As with the mid-span deflection comparisons, further modification of the model was not considered due to time constraints. The effect of modifying the end restraint conditions could be another area where further research could be undertaken.

6.2.3. Differential Deflections

As the simple grillage model did not have any differential girder-deck deflections or differential deck deflections, no comparison can be made with the field testing results.

6.2.4. Load-Deflection Comparisons

Time constraints did not allow for the different loading conditions between phase 1 loading and phase 2 loading to be modelled and analysed. No comparisons were made between the field testing results and the FEA results.

6.3. Strains

6.3.1. Girder Strains

After the FEA of the simple grillage model was completed, axial girder strains were only predicted to be in the order of 5 – 10 μ. As the field test results showed axial girders strains in the order of 300 – 400 μ, there is no justification in comparing these results.

As with the deflection comparisons, modifying the end restraints of the model to full moment connections at the abutments should give more realistic results. Again, time constraints excluded the model modification in this area, and no further comparison between field testing axial girder strain results and FEA axial girder strain results took place.

6.3.2. Deck Strains

No deck strains were predicted from the finite element analysis of the simple grillage model. Accordingly, no comparisons could be made between the field testing results and the FEA results. As this project is predominantly concerned with the behaviour of the girders, the lack of data for analysis is of no great significance.

6.3.3. Shear Strains

The Strand7 FEA software package does not allow for the measurement of shear strains in simple grillage model analysis. A much more detailed model must be constructed before shear strains can be predicted using the Strand7 software package. As with other strain comparisons, no comparison could be made between the field testing results and the FEA results.

6.4. Summary

The main issue involved in making comparisons between field testing results and FEA results is the lack of FEA data to compare with field data, particularly strains. This appears to be mostly due to the set up of the end restraints in the simple grillage model. Time constraints precluded the modification of the end restraints in the modelling of the bridge.

The comparison between the maximum mid-span girder deflections showed that the FEA results were greater than the field testing results, which is in agreement with historical FEA results. The magnitude of the difference was unusually high; this may be due to the girder-deck interaction, which can be difficult to model using Strand7 software. The end restraint conditions may also have had some effect on the mid-span deflections, so modifying the end restraint conditions could affect the predicted maximum mid-span deflections of the simple grillage model.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1. Introduction

This research project has investigated the behaviour of fibre composite beams under different types of loading in a composite bridge structure. The project also investigated the effect of modification of transverse stiffeners (diaphragms) using the Strand7 FEA software package, and whether there was any need for the inclusion of diaphragms in the construction of the full-size bridge.

Different models were developed for finite element analysis and comparisons were made between the different models to determine whether a simple grillage model could be used to accurately analyse the composite bridge structure, including the accurate prediction of maximum mid-span deflections.

After analysing FEA data and deciding which model to use for comparison with field testing results, data from both field testing and FEA were compared and discussed.

The project also investigated issues involved in increasing the span of the bridge to full-scale. Using Strand7 software, a simple grillage model was developed and analysed, and results discussed.

7.2. Achievement of Objectives

To satisfy the aims of the project, the objectives set out in Chapter 1 needed to be achieved. This section shows how each objective was achieved.

1. *Research the background information on previous field testing and instrument placement of bridge structures.*

Chapter 2 covers this objective, as a literature review was conducted and involved literature from a number of sources (online, journals, books, government publications, etc.) relevant to the various areas that needed to be researched.

2. *Develop a testing plan including placement of instrumentation on the beams and deck of the bridge, and static and live loading of the bridge.*

This objective was achieved, as Chapter 3 demonstrates. The methodology of the testing plan, instrument placement, and loading plans were covered in this chapter.

3. *Collect data from field testing of the bridge, as appropriate.*

The testing was carried out according to the testing plan, and field data was collected from the testing. All of the collected data can be found in Appendix D as Microsoft Excel worksheets.

4. *Analyse field data for use by Department of Main Roads, and compare field data with analysis using appropriate finite element software package (Strand7), taking deck effects into account.*

The analysis of the field testing data collected is discussed in Chapter 4, and the finite element analysis is discussed in Chapter 5, with comparisons of the two analyses being discussed in Chapter 6.

5. *Determine the viability of using simplified analysis methods (grillage analysis) to predict deflections accurately.*

Chapter 5 discusses the implications and limitations of using simplified analysis methods to predict deflections accurately. As finite element analysis is historically conservative, the results from analysis of the grillage model supported this evaluation, and could be used to predict conservative deflections reasonably accurately. More detailed member properties would have helped in developing a more realistic model, and therefore more realistic predicted deflections, but for the purpose of the project, use of simplified analysis methods appear to be viable.

6. *Investigate the issues involved in increasing the span of the bridge to full size.*

This objective is also discussed in Chapter 5, and a number of issues arose from the development and finite element analysis of the full-size bridge model. Predicted deflections were greater than the limiting value of span/500, so further work would need to be conducted to reduce predicted deflections below this limiting condition. These include modification of the beam and diaphragm parameters (E value, dimensions) and modification of the end restraint conditions, but no further research was undertaken in this area.

Given time, conduct a cost-benefit analysis into the viability of replacing hardwood timber bridge beams with fibre-reinforced polymer beams.

As the field testing was conducted much later than originally expected (late July instead of early May), time constraints precluded the achievement of this objective. This area of study could be possibly undertaken in future projects dealing with the production and use of fibre composite members in the transport industry.

7.3. Conclusions

The results from the field testing showed that the girder mid-span deflections initially predicted from the preliminary truck testing were much greater than the recorded mid-span deflections from the formal testing. It can be concluded from this that either the estimated rear axle load of 30 kN was less than the truck actually carried, the deflection of the girders were not proportional to the load being carried, or the girder-deck interaction had more effect when greater loading was involved. More truck runs with different accurately measured load conditions should be able to answer this effectively.

As expected, the mid-span deflection of the girders was found to be the most critical parameter, as other deflections measured appeared to be structurally insignificant, and all strains measured were much smaller than the yield strains of the members. The difference between the dynamic loading deflections and the static loading deflections could have some effect when compared with the natural frequency of the bridge; however at the time of writing, natural frequency field testing had not taken place.

The magnitude of the difference between the field testing maximum mid-span deflections and the finite element analysis maximum mid-span deflections was larger than expected. A more detailed model and modification of various parameters would give a better understanding of where possible errors in the developed bridge model may exist. Due to time constraints, it was not possible to undertake this for the project, but could be considered in further study.

It was determined that even though the deflections predicted by the finite element analysis were larger than expected, a simple grillage model could be used to predict deflections reasonably accurately but conservatively. The addition of the deck to the model did not alter the results sufficiently to warrant the extra time taken in developing and analysing the model.

Only preliminary investigation was undertaken when developing and analysing the full size bridge model. To get any realistic predictions, the model would need to be developed using accurate parameters (E values, member dimensions, end restraint conditions etc.) and then analysed. Based on the results comparison from this project, a simple grillage model should produce reasonably accurate predicted deflections.

7.4. Recommendations

Recommendations for further studies include:

- Conducting further field testing of the bridge using different accurately measured loading conditions to investigate the load-deflection linearity of the girders.
- Further field testing could also be conducted after the removal of the diaphragms to investigate the difference in deflections with and without diaphragms.
- Undertaking a cost-benefit analysis to determine the viability of replacing hardwood timber and concrete bridge members with fibre composite members.
- Modification of the Strand7 finite element model parameters (E values, member dimensions, end restraint conditions) to investigate what effect each parameter has on predicted deflections.
- As discussed in Chapter 2, conducting a survey to discover the availability of courses directly relating to FRP research and development. The increase in the number of courses should lead to the development of standards and specifications which could then be used for FRP production to be used in the transport industry, and ultimately all industries.

7.5. Summary

As the need for rehabilitation of existing bridges becomes more widespread, the use of fibre composite members to replace hardwood and concrete members in existing bridges and in the construction of new bridges appears to be justified. Once increased production leads to lower production costs, the ease of transport and installation of fibre composite members should give these products a distinct cost advantage over hardwood and concrete members.

The analysis of the fibre composite beams for the half-size test bridge has shown that these beams comply with the limiting maximum mid-span deflection of span/500 even though the loading used in the field testing was greater than the legal axle load limit. This shows that fibre composite members are viable in the rehabilitation of existing bridges and construction of new bridges.

For this project, the use of simplified analysis methods using finite element analysis software was justified, as increasing the complexity of the model did not alter the predicted deflections sufficiently to justify the extra time involved in developing the model. Depending on the complexity of the construction being undertaken, the need for more detailed modelling may become necessary for the achievement of realistic predictions.

The use of fibre composite beams in bridge construction depends on the implementation of specific standards for fibre composite materials. As there is an increasingly greater amount of research being conducted worldwide into the properties and uses of fibre composites, these standards should be being developed shortly.

With the increased need for sustainable construction, it is important to realise that if there is a reduced demand on timber and concrete for civil infrastructure, the use of fibre composite materials can only increase the sustainability of civil construction, particularly in the transport industry. This project has shown that the use of fibre composite materials for bridge construction/rehabilitation is structurally viable, and should become economically and socially viable in the future.

References

Australian Building Codes Board, 2007, *Building Code of Australia 2007 Edition.*

Austroads Guide AP-T23: Dynamic Interaction of Vehicles and Bridges, 2003, Austroads Incorporated, Sydney.

Ayer, S. & Van Erp, G. 2002, "A code of practice for the structural design of fibre composites", *Structures & Buildings*, 157 Issue SB1, January 2004, p 3-8

CRC for Construction Innovation, 2002, *Case Study: Comparison of current QDMR practices and applicability of FRP technology,* Research Program C: Delivery and Management of Built Assets, Queensland University of Technology (QUT), Brisbane.

CRC for Construction Innovation,2004, *Innovation Case Study No 5: Australia's First Fibre-Reinforced Polymer Bridge Deck on the Road Network*, BRITE Project, Queensland University of Technology (QUT), Brisbane.

Dunn, M. et al. 2005, "Tama County's Steel Free Bridge Deck", *Proceedings of the 2005 Mid-Continent Transportation Research Symposium,* Ames, Iowa, August 2005.

Fibre Composite Projects Technical Note 54 March 2006, Queensland Government Department of Main Roads.

Fu, C C. & Harwood, K. 1999, "Practice of Restoring a Damaged Historical Truss Bridge", *Practical Periodical on Structural Design and Construction,* Vol 5, Issue 3, August 2000, p 122-125.

Hybrid beams for sustainable replacement of hardwood timber in bridge construction, 2006, Australian Government – Forest and Wood Products Research and Development Corporation (FWPRDC), Melbourne.

Hybrid Concrete/FRP Bridge Development, 2005, 2005 Nova Award Nomination 10, Construction Innovation Forum, Canton, MI.

Hollaway, L.C. & Head, P.R. 2001, *Advanced Polymer Composites and Polymers in the Civil Infrastructure*, Elsevier Science Ltd, Oxford.

Jenkins, D. 2004, *Bridge Deck Behaviour Revisited*, Austroads Bridge Conference, Hobart 2004.

Konda, T.F. 2003, “Construction and Evaluation of the First Modified Beam-in-Slab Bridge”, *Transportation Scholars Conference 2003*, Midwest Transportation Consortium (MTC), Iowa State University.

Lowe,S. 1999, *Collapse Behaviour of Reinforced Concrete Beam and Slab Bridges*, MPhil Dissertation, Cambridge University.

Mirmiran, A. et al. 2002, “World Survey of Civil Engineering Programs on Fiber Reinforced Polymer Composites for Construction”, *Journal of Professional Issues in Engineering Education and Practice*, ASCE, July 2003

Moses, F., Lebet, J.P. & Bez, R. 1992,” Applications of Field Testing to Bridge Evaluation, *Journal of Structural Engineering*, Vol 120, No 4, June 1994.

Potisuk, T. & Higgins, C. 2005, “Field Testing and Analysis of CRC Deck Girder Bridges”, *Journal of Bridge Engineering*, Vol 12, No 1, January 2007, p53-63.

SAA, 2001, *AS 1145.4-2001: Determination of tensile properties of plastics materials - Test conditions for isotropic and orthotropic fibre-reinforced plastic composites.*

SAA, 2001, *AS 1145.5-2001: Determination of tensile properties of plastics materials - Test conditions for fibre-reinforced plastic composites.*

Samman, M., Sennah, K. & Kennedy, J.B. 2001, “Positioning of bearings for curved continuous spread-box girder bridges”, *Canadian Journal of Civil Engineering*, Vol 29, 2002, p 641-652

Shenoi, R.A., Moy, S.S.J. & Hollaway, L.C. 2002, *Advanced polymer composites for structural applications in construction*, Thomas Telford Publishing, London.

Tan, G. H., Brameld, G. H., and Thambiratnam, D. P., “Development of an analytical model for treating bridge-vehicle interaction”, *Engineering Structures*, Vol. 20, Nos 1- 2, pp 54-61, 1998

Van Erp, G. et al. 2002, *An Australian approach to fibre composite bridges*, University of Southern Queensland.

Watkins, S.E. et al. 2001,” Instrumentation of a smart composite bridge for short-span applications”, *8th Annual International Symposium on Smart structures and Materials: Smart Systems for Bridges, Structures and Highways*, 4-8 March 2001, Newport Beach, CA

APPENDIX A - Project Specification

University of Southern Queensland

Faculty of Engineering and Surveying

ENG 4111/2 Research Project

PROJECT SPECIFICATION

FOR:	**Christopher GREEN**
TOPIC:	Testing and analysis of fibre composite beams in a bridge structure
SUPERVISORS:	Thiru Aravinthan Karu Karunasena
SPONSORSHIP:	Department of Main Roads, Queensland Government
PROJECT AIMS:	This project aims to investigate the effects of various forms of loading on fibre composite beams in a bridge structure, to determine whether simplified methods can be used to analyse the composite structure, and to investigate issues involved in increasing the span of the bridge to full scale.

PROGRAMME: **Issue B, 10 May 2007**

1. Research the background information on previous field testing and instrument placement of bridge structures.
2. Develop a testing plan including placement of instrumentation on the beams and deck of the bridge, and static and live loading of the bridge.
3. Collect data from field testing of the bridge, as appropriate.
4. Analyse field data for use by Department of Main Roads, and compare field data with analysis using appropriate finite element software package (Strand7), taking deck effects into account.
5. Determine the viability of using simplified analysis methods (grillage analysis) to predict deflections accurately.

6. Investigate the issues involved in increasing the span of the bridge to full scale.
7. Given time, conduct a cost-benefit analysis into the viability of replacing hardwood timber bridge beams with fibre-reinforced polymer beams.

AGREED: ______________________ (Student) _____________________ (Supervisor)

Co-examiner: _____________________ Dated ___/___/___

APPENDIX B - Test Bridge Site and Specifications

I. Test Bridge Site

The test bridge site is at the north-western end of Handley Street, on USQ grounds.

II. Test Bridge Specifications

The test bridge has a span of 10 m and is 5 m wide. The girders are approximately 400 mm square; the diaphragms are bolted to brackets attached to the girders, and are approximately 250 mm wide and 400 mm deep. There are eight deck panels, each of which is approximately 1200 mm wide and 120 mm deep. The girders are bolted to concrete abutments.

Figure B-1: Girder and Diaphragm Layout

Figure B-2: Installation of Deck Panels

Figure B-3: Completed Bridge

Figure B-4: Bridge marked for Testing Runs

APPENDIX C – Risk Assessment

Hazard	Likelihood	Exposure	Consequences	Control
Burns caused by soldering iron when soldering plugs onto instrument leads	Substantial	Occasionally	Minor equipment damage Minor injury	Care and attention Know how to use equipment Wear appropriate clothing/ eye protection
Personal injury caused by machinery when making base plates for string pot displacement gauges	Substantial	Rarely	Minor equipment damage Minor/major injury	Care and attention Know how to use equipment Wear appropriate clothing/ eye protection
Skin contact with epoxy glue when attaching strain gauges to bridge	Substantial	Rarely	Minor equipment damage Minor injury	Care and attention Wear appropriate clothing (gloves)

Injury caused by truck in field testing	Substantial	Rarely	Major equipment damage Major injury Possible death	Stay clear of moving vehicle Wear appropriate clothing (boots)
Heatstroke/Dehydration	Slight	Rarely	Minor/major illness	Wear appropriate clothing (hat, long sleeves/sunscreen)
Eye fatigue/Headaches from computer use	Substantial	Frequently	Minor illness	Take regular breaks from computer
Loss of data caused by computer malfunction	Substantial	Frequently	Loss of data	Save frequently Back up data on separate disk

APPENDIX D – Field Testing Data Worksheets and Strand7 FEA files

I. Field Testing Data Worksheets

All field testing data can be found as supplementary files in the subdirectory named **ChristopherGREEN_appendixDworksheets** on the CD inside the back cover. Worksheets include: data from preliminary truck trials conducted on 6 July 2007, formal testing conducted on 24 July 2007, and analysis of Strand7 FEA results.

II. Strand7 FEA Files

All Strand7 FEA models, including modifications, can be found as supplementary files in the subdirectory named **ChristopherGREEN_appendixDstrand7files** on the CD inside the back cover. Models include: initial grillage model, diaphragm modification, deck addition (both approaches), and initial full-size bridge model.