

ARCHFORM DESIGN AND TESTING OF A STEEL FREE DECK CONSTRUCTED USING PARTIAL DEPTH PRECAST CONCRETE PANELS

BY

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A Thesis submitted to
The Faculty of Graduate Studies
In Partial Fulfilment of the Requirements for the Degree of

MASTER OF SCIENCE

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Winnipeg, Manitoba

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Manitoba in partial fulfillment of the requirement of the degree**

Master Of Science

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ABSTRACT

Transportation Authorities are looking for innovative methods to increase the service life of bridge decks and to reduce construction schedules and delays to road network users during maintenance and reconstruction works. Therefore, the ArchForm partial depth precast concrete panel concept was developed. It combines precast panel technology with the steel free deck concept. The goal of this system is to provide a cost effective steel free deck system that can be constructed using partial depth precast panels.

This project summarises the design of the ArchForm panel and an experimental testing program undertaken at the University of Manitoba. The testing program was divided into two phases, the first to test partial depth precast concrete panels and the second to test a complete deck system constructed using the partial depth precast concrete panels. The first phase of testing verified that the panels could withstand construction loading which included their self weight, weight of the upper portion of the deck concrete cast on site and live load during construction. In addition, it verified that the panel construction was robust and would not be easily damaged during construction. The second phase of testing of the deck system included three static load tests. Full scale models were used in both testing phases.

The results of this program confirm that a steel free deck system constructed using partial depth precast concrete panels performs as well as a traditional steel free deck cast monolithically when loaded under static axle loading.

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Appendix 1 – Panel fabrication drawings for Phase 1 and 2 Testing

Appendix 2 – Results from PUNCH Analysis

LIST OF SYMBOLS

A	=	Cross sectional area of a steel strap (mm^2)
A_{sc}	=	Cross section area of a shear connector (mm^2)
B	=	Diameter of idealised load area (mm)
β_1	=	Concrete stress block variable
d	=	Depth of concrete slab (mm)
E_c	=	Concrete Modulus of Elasticity (MPa)
E	=	Modulus of Elasticity of Steel (200 GPa)
f'_c	=	28 Compressive Concrete Strength (MPa)
F_s	=	Factor given in CHBDC Clause 16.7 (g)
F_u	=	Ultimate Force of the shear connector steel (kN)
q_{sr}	=	Resistance force of the shear connectors (kN)
S_g	=	Centre to centre spacing of beams supporting a deck slab (m)
Sprx	=	distributed linear spring constant for girders
S_l	=	Spacing of steel straps (m)
t	=	Thickness of deck slab (mm)
V_{strap}	=	Shear force in the steel strap (N)
ϕ_{sc}	=	Material reduction factor for a shear connector = 0.85

1.0 INTRODUCTION

1.1 GENERAL

Corrosion of reinforcing steel in bridge decks leads to premature bridge deck failures resulting in increased maintenance and/or reconstruction. Over the next ten years maintenance costs and capital costs related to corrosion is estimated to be between \$1.07 billion to \$2.93 billion in the United States (Yunovich et al, 2005). Since bridges represent vital transportation links in any transportation network, not only are the maintenance and/or reconstruction costs significant; these activities disrupt a transportation system. The cost of disruption is estimated to be as much as ten times more than the capital construction cost based on a life cycle cost analysis (Yunovich et al, 2005). The cost of disruption is defined by the indirect costs to transportation network users resulting from increased travel times and/or travel distances, (Yunovich et al, 2005). To reduce these costs Transportation Authorities are seeking new technologies to increase the lifespan of bridge decks and to minimise lane and bridge closures during reconstruction activities.

One possible solution is to use a steel free bridge deck that is constructed using partial depth precast concrete panels, coined the ArchForm system. In this system, the service life of the bridge deck is increased by utilising the steel free deck concept which only requires crack control reinforcement within the bridge deck. The amount of reinforcement is substantially less than a bridge deck designed using traditional methods and in cases, a non-corrodible reinforcement is used. Utilising partial depth precast concrete panels reduces the amount of on-site labour by eliminating the time required to install and strip deck formwork.

1.2 BACKGROUND

Road salts are the most significant factor contributing to premature bridge deck failure. The corrosion rate of reinforcing steel greatly increases when the concentration of chlorides has reached a point when the protective passive layer around the rebar begins to be destroyed. This passive layer has a high pH that develops when wet concrete reacts with reinforcing steel. Once chlorides permeate through the concrete and reach a concentration referred to as the chloride threshold, the protective layer around the steel reinforcement is broken down and corrosion occurs.

In recent years, engineers and transportation agencies have tried to prolong the service lives of bridge decks by using one or a combination of the following:

- Increasing the clear concrete cover to the upper mat of reinforcing steel
- Using high density, low permeability concrete to slow the infiltration of chlorides
- Using waterproof membranes or epoxy overlays to prevent chlorides from entering the concrete
- Using different coatings such as galvanizing or epoxy on the steel reinforcement to slow the rate of corrosion of the reinforcement
- Using a variety of different materials or alloys like stainless steel, stainless clad rebar, MMFX rebar, glass fibre rebar which have better corrosion resistant properties compared with black steel
- Using additives in concrete mixes to inhibit corrosion

The goal of these measures is to either alter the reinforcement properties or environment surrounding the reinforcement to mitigate corrosion. Each measure increases the service

life of bridge decks with varying degrees of success. An extremely effective solution is to reduce the amount of reinforcement within the bridge deck.

Conventional bridge decks are designed and constructed using an orthogonal grid of reinforcing steel in the upper and lower portions of the bridge deck (Figure 1-1). Design is normally based on an empirical method that accounts for arching behaviour in the deck. This design method allows the engineer to reduce the amount of transverse rebar compared with that required assuming flexural behaviour of the slab. Extensive research on the behaviour of bridge decks has proven that decks will fail in punching shear provided that adequate transverse and longitudinal restraint is provided (Jaeger et al, 1995). This discovery has led to the development of the steel free deck which eliminates all internal reinforcement and provides transverse restraint by external straps at the underside of the slab.

The original steel free deck system utilised chopped polypropylene fibres within the concrete to control potential cracking from plastic shrinkage and variations in temperature with no internal reinforcement. A layer of non-corrodible reinforcement is now utilised in steel free decks to provide crack control and to prevent a full depth crack from forming. The amount of crack control reinforcement is much smaller than that calculated using traditional deck design methods. Therefore, the higher material cost of non-corrodible reinforcement compared with black rebar has less impact on the total reinforcement cost in a steel free deck compared to a conventional deck.

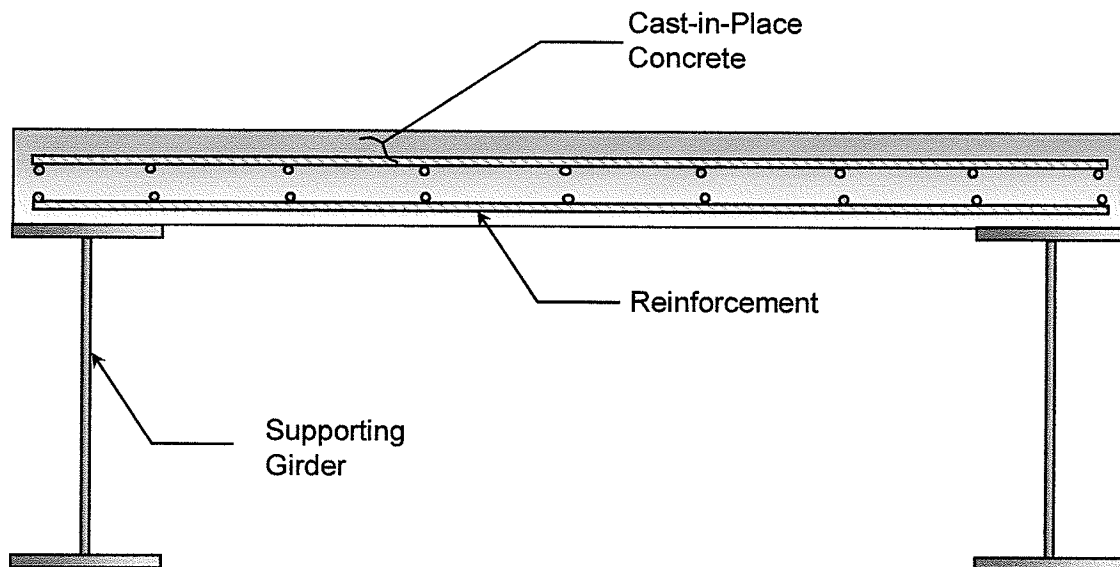


Figure 1-1. Typical Orthogonal Bridge Deck Reinforcement in Two Layers.

ARCHFORM CONCEPT

The ArchForm system uses partial depth precast concrete panels to construct a steel free deck. In effect, the slab is cut in half on horizontal plane, forming two elements. The lower element is the partial depth precast panel which is constructed with a bottom mat of crack control reinforcement and the steel straps required for the steel free deck system (Figure 1-2 and 1-3). The upper half consists of the cast in place concrete with no reinforcement.

The panels are pre-fabricated, delivered to the construction site and placed on top of the girders between the girder lines. They function as a stay-in-place form for the cast-in-place upper deck concrete. Forming is required at the longitudinal ends of the structure and may or may not be required at a cantilever depending on its width. Concrete is cast on top of the panels to form a complete bridge deck. Continuity between the panels and the cast in place portion of the bridge deck is achieved by roughening the top surface of the panels to

improve bond and by extending the steel straps with shear connectors over the girder flange that also has shear connectors.

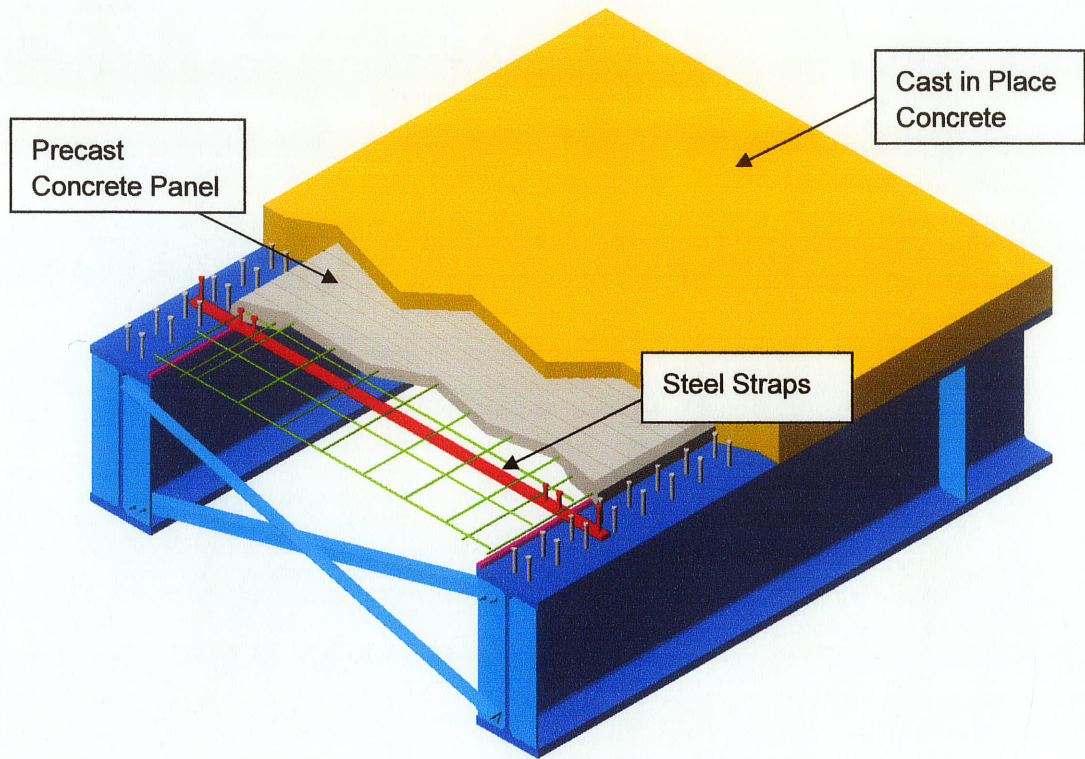


Figure 1-2. ArchForm Bridge Deck Showing the Precast Panels and Completed Deck Concrete.

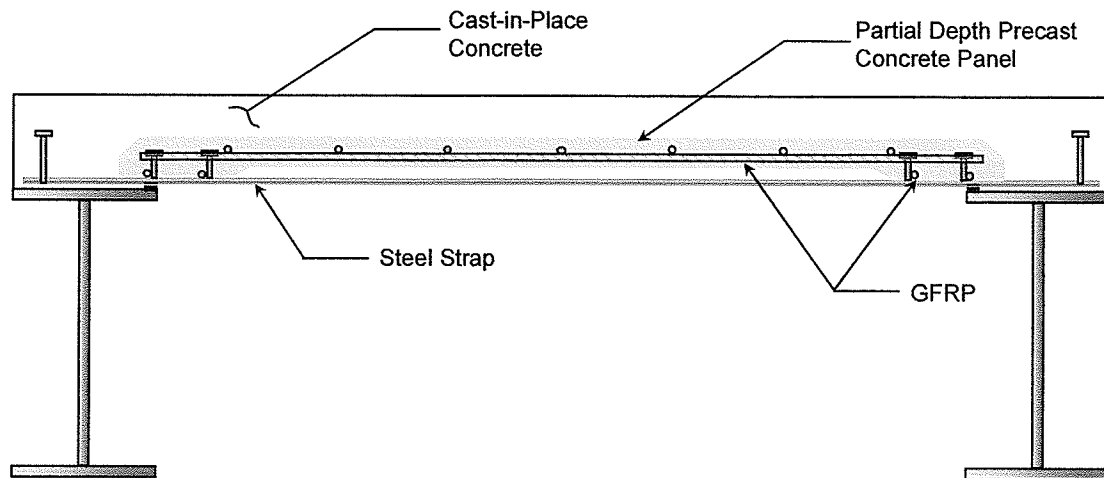


Figure 1-3. ArchForm Bridge Deck Cross Section.

1.3 OBJECTIVE AND SCOPE

The objective and scope of the research program was to:

- investigate the current state of the art of partial depth precast concrete panels and the steel free deck system
- determine construction loading on partial depth precast concrete panels outlined in the various design codes
- design the partial depth precast concrete panels and prove that they can safely resist construction loading
- design the steel free deck system constructed using partial depth precast concrete panels and prove that it performs as well as or better than a traditional system cast monolithically

2.0 LITERATURE REVIEW

2.1 STEEL FREE DECKS

2.1.1 INTRODUCTION

Bridge decks have traditionally been designed and reinforced assuming that they behave and fail in flexure (Jaegar et al, 1995). The primary behaviour has been assumed to be transverse bending between supporting girders. Experimental testing and field performance over the past 30 years has shown that deck slabs will fail in punching shear rather than in bending. When this occurs, the deck develops compressive membrane forces and the ultimate capacity is many times more than the flexural capacity (Jaegar et al, 1995).

The steel free bridge deck design relies on internal arching of the deck slab to resist traffic loads. In 1976, the Ontario Bridge Design Code (OHBDC) was the first design code in the world to introduce a deck design method accounting for internal arching action in reinforced concrete bridge decks (Jaegar et al , 1995). This design method has since been adopted by the American Association of Highway and Transportation Organisation (AASHTO), the Canadian Highway and Bridge Design Code (CHBDC) in 2001 and by many other bridge design codes. This design method reduces the amount of reinforcing steel required compared to traditional bending theory, resulting in a significant savings in construction costs and improved fatigue performance.

Research by Mufti et al in 1993 demonstrated that a deck slab made composite with the supporting girders will fail by punching shear, provided that there is adequate transverse

and longitudinal constraint (Mufti et al, 1993). This led to Mufti et al's development of what is now called the "steel free deck". An external steel strap is used to restrain the deck transversely, as shown in Figure 2-1 and no internal reinforcement is required. Longitudinal restraint is provided by the axial stiffness of the girders. However, when axle loads are applied near the ends of the girder, the girders do not provide effective longitudinal restraint of the deck. Therefore, edge stiffening beams are used.

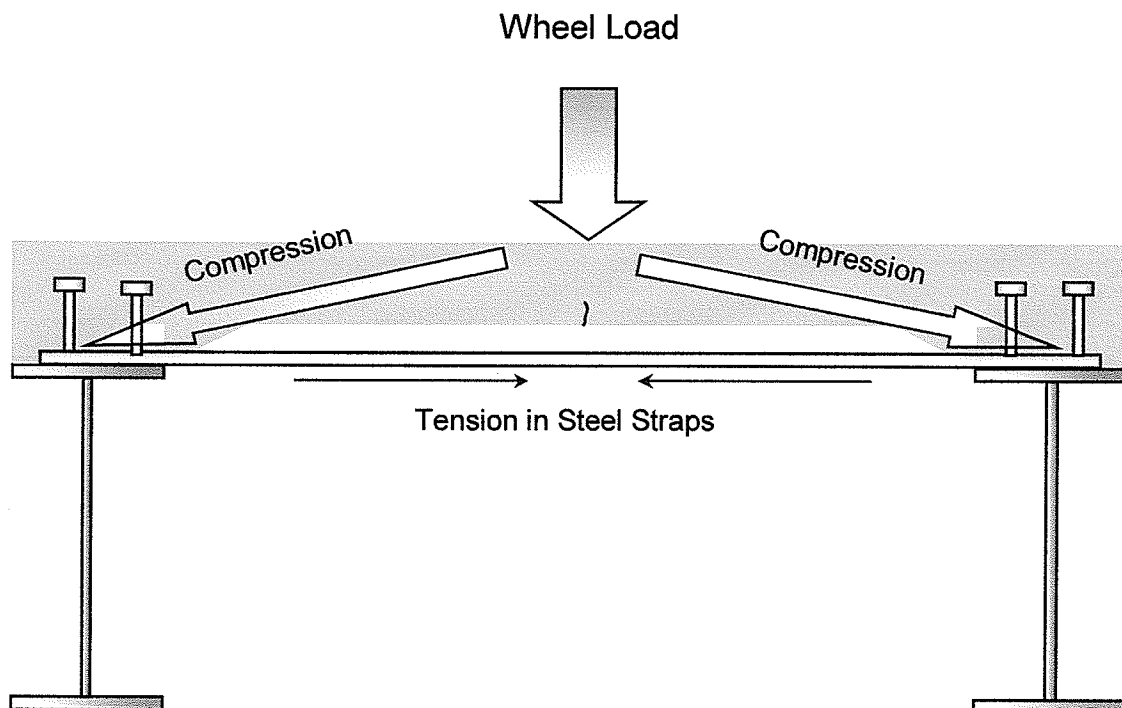


Figure 2-1. Steel Free Deck Concept.

This system, coined the "first generation steel free deck", was constructed using external steel straps made composite with the top flange of the girders. Composite action was achieved by welding the steel strap to the supporting steel girder's top flange. Shear connectors were welded to the girder's top flange similar to the typical composite design methodology. In concrete girders, shear connectors are welded to the top of the steel straps

that are located above the concrete girder. This detail is also used on steel girders and is shown in Figure 2-1.

This section provides a review of the history, description of the design, and present state of the art related to the steel free deck slabs.

2.1.2 *HISTORY OF STEEL FREE DECK SLABS*

There are presently two generations of steel free decks. The first generation steel free deck does not contain internal reinforcement and uses fibre reinforced concrete to control shrinkage and temperature cracking. Full depth longitudinal cracks were discovered in all of these decks shortly after construction. The cracks have proven to be only unsightly, and do not affect the structural integrity of the deck. The second generation steel free deck was invented to eliminate the full depth cracks. The second generation steel free deck utilises a grid of crack control reinforcement to reduce the crack widths of longitudinal cracks and to ensure that these cracks do not become full depth cracks. The following describes the field performance of steel free decks constructed in Canada and laboratory fatigue testing.

Five first generation steel free decks have been constructed on Canadian highway bridges between 1995 and 1999 and one on a wharf structure (Mufti et al, 1999). Shortly after construction on the five bridge decks, a 1 mm wide longitudinal crack was observed at the soffit's midspan of the deck between girder lines. The cracks were an indication that the steel straps were resisting load and that arch action rather than bending behaviour occurred. However, the wide crack raised concerns about the long term structural and service performance of the decks. Engineers were concerned whether there was sufficient aggregate interlock on either side of the crack to transfer the vertical portion of the loads over the life of the deck, and, in the case where no secondary protection on the top of the

deck like a waterproof membrane complete with an asphalt overlay was provided, would localised concrete deterioration occur at this crack.

All of these structures have been monitored visually and some have been load tested. Interviews with individuals intimate with each of these structures in December of 2004, indicated that these decks continue to safely resist truck loading (Brett, 2004 and Mufti, 2004). There have been no indications that the 1 mm wide cracks have increased in width or caused any other distress to the deck since the cracks were first observed. Besides monitoring the actual performance of these structures, fatigue testing in the laboratory has been carried out.

Extensive laboratory testing has been performed to better understand the full depth cracks to ensure that these decks can safely support truck loading and to develop a method to eliminate full depth cracking. Research by Mufti et al (2001) showed that a full depth crack can form through the deck slab under pulsating loads, or loads simulating a moving vehicle. This occurs since the compressive force regime exists in the slab only as long as the wheel load is above the crack. As a wheel passes over the crack, the slab rebounds at a given location and tensile forces are induced at the sharp tip of the crack. The tip gradually rises until it has formed a full depth crack.

At Hall's Harbour Wharf in Nova Scotia, an existing wharf was rehabilitated with a steel free deck in 1998 (Newhook et al, 2001). A mesh of GFRP reinforcement was installed in a steel free deck which contained polypropylene reinforcement. A single longitudinal crack was observed at the apex of the soffit about eight months after construction. Careful examination of the structure revealed that the crack did not extend through the slab, confirming that a layer of reinforcement in the deck slab arrests crack propagation through the slab thickness.

Fatigue testing by Mufti et al (2001) has also shown that a grid of reinforcement placed at the bottom of the steel free deck controls the widths of cracks. The first steel free bridge deck incorporating this technology was constructed on the North Perimeter Highway over the Red River in Winnipeg, Manitoba, Canada. The most westerly land span of the 10 span westbound structure was retrofitted with the second generation steel free deck in the fall of 2003. All spans consist of haunched steel plate girders. All other spans were constructed using a conventional bridge deck with a grid of upper of lower reinforcing steel.

A grid of glass fibre reinforcement was placed at the top and bottom of the steel free deck slab to provide crack control and flexural reinforcement at the deck cantilevers. A field investigation of the steel free deck in September, 2004, revealed that single longitudinal cracks were present at some locations near the mid span of the deck between girder lines. It could not be determined with absolute certainty whether or not the cracks were full depth because the bridge deck was covered with an asphalt overlay. No efflorescence was observed at the soffit and the crack widths were hairline. Therefore, it is unlikely that the cracks were full depth crack.

2.1.3 *DESIGN PARAMETERS*

The following steel free deck design aspects are discussed below:

- Spacing of steel straps and dimensions
- Connection of concrete barriers
- Design requirements for cantilevers
- Design requirements for continuous spans.
- Edge stiffening
- Crack control

Work by Newhook (1997) determined that the most efficient strap spacing was 1000 mm to 1200 mm. Typically, this results in a steel strap with cross sectional dimensions of approximately 25 mm x 50 mm. The thickness of the strap is recommended to be at least 25 mm to minimise strap vibrations under truck loading. Straps measuring 14 mm x 100 mm were used on the Salmon River Bridge and were found to vibrate under truck loading (Newhook, 1997).

Concrete barriers utilising a stainless steel anchorage and a grid of GFRP reinforcement are typically used with the steel free deck. These materials are used because they will provide longer service lives than black reinforcing steel when exposed to chlorides. Testing by Maheu and Bakht (1994) proved that barriers connected using a double headed tension bar provide an adequate connection for the barrier to a deck (Maheu et al, 1994).

Typically, continuous bridges are preferred by transportation authorities since they minimise the number of expansion joints required and associated maintenance of these joints, provide a much better riding surface, and reduce girder moments and deflections. No reinforcing steel is required in the upper portion of the concrete slab with the exception of negative moment regions occurring over the piers on continuous structures and at cantilevers. An upper layer of reinforcement is required in these circumstances and traditional flexural design methods are used.

Edge stiffening is required to ensure adequate longitudinal restraint in steel free decks (Newhook et al, 2001). This ensures a punching shear failure when an axle load is applied near the end of a bridge span. The CHBDC currently provides four edge stiffening details, including two reinforced concrete options and two steel channel options. Any option may be the most economical depending on the location and geometric constraints of a particular bridge,

Bridges are exposed to changes in temperature and humidity which generate tensile forces in concrete. Thermal stresses are generated by temperature changes and differential temperature effects. Bearings are designed to allow a bridge to expand and contract without any constraints. During expansion and contraction, thermal forces are generated by friction between the bearings and supporting girders. Differential thermal stresses occur because the temperatures in deck elements and girder elements vary. The temperature variation is greatest on hot sunny days when the top portion of the deck is rapidly heated by the sun and the shaded portions of the deck and girders are not.

Concrete tends to shrink over time as it dries inducing tensile stresses in the concrete. Normally, crack control reinforcement is provided to mitigate the effects of temperature and shrinkage cracking. Since the steel free deck concept has attempted to eliminate internal reinforcement, significant research has been performed on the use of fibre reinforced concrete to control temperature and shrinkage cracks. Polypropylene fibres added to this concrete have proven to increase the tensile capacity of the concrete to mitigate cracking (Khanna et al, 2000). This technology has been successfully used in many structural engineering applications.

2.1.4 *FATIGUE RESISTANCE*

Experimental testing has proven that the capacity of steel free bridge decks under static loading is many times the actual axle loading. Fatigue tests are a much better judge of their worthiness because they simulate the cyclic truck loading that decks experience. Fatigue testing on the steel free deck has been used to understand the response of the first generation and second generation steel free decks. The three types of dynamic tests that have been performed on steel free deck slabs are:

- Single Moving Wheel
- Sequential Moving Wheel
- Single Pulsating Wheel Load

In a single moving wheel test, a wheel with a known weight is moved across the deck at a given speed. This test best represents actual wheel loads. The sequential moving wheel test uses a series of load pads at fixed locations. Actuators are used to apply loads to the pads in sequence, simulating a moving truck load. In the single pulsating wheel load test a single load pad is loaded and unloaded. This test provides the least accurate representation of actual moving loads compared with the other two methods, but is the simplest and most economical to perform.

FIRST GENERATION STEEL FREE DECK

After the full depth cracks were discovered at the soffit of the first generation steel free decks, fatigue tests were performed to determine the effect the crack will have on the long term structural performance of the deck. Rolling wheel load tests were conducted in Japan by Matsui et al (2001) on three different steel free deck slabs containing polypropylene fibres. Matsui concluded that the steel free deck slabs exhibit significantly higher fatigue resistance than conventional reinforced concrete slabs. In addition, the crack patterns observed consisted mainly of longitudinal cracks which bifurcated into a Y pattern at the transverse free edge. The bifurcation point is the weak point of the deck.

Fatigue tests using single pulsating wheel loads were conducted at the University of British Columbia in 2002 on the full depth precast concrete panels used to construct the Lindquist Bridge (Mufti, 2001). Two panels measuring 4.3 m wide by 3 m long were constructed using fibrillated polypropylene fibres at volume fractions of 0.1% and 0.4%.

Both panels were tested under pulsating loads peaking at 140 kN. Panel 1 with 0.1% fibres, was subjected to 370,000 cycles and panel 2 with 0.4% fibres was subjected to 532,000 cycles. A full depth crack developed in both panels. Failure loads were at 454 kN and 500 kN in panels one and two respectively. The authors note that a half million cycles of 140 kN wheel load is more severe than the required two million cycles of 75 kN wheel load according to the Miner's rule. This testing helped confirm that even if a full depth crack occurs the static capacity of the steel free deck under nearly a half million load cycles is greater than twice the factored axle load.

The CHBDC does not provide explicit guidance on fatigue testing methods for deck slabs (Mufti, 2002). Therefore, a statistical model has been developed to correlate the predicted actual lifetime loading on a bridge deck to an axle load of known weight applied a given number of times. This has been calculated assuming that the bridge deck is in service for 75 years, the maximum possible average annual daily traffic is 20,000 vehicles per lane, and there is a maximum of 20% truck traffic (refer to commentary clause C3.6.1.4.2 of the 1998 AASHTO LRFD specification).

The relationship which considers the varying magnitude of axle loads has been specifically developed for steel free deck slabs. It enables one to determine the load magnitude and number of load cycles for fatigue loading. Based on this relationship, fatigue testing under 25 t (245 kN) and 50 t (490 kN) loads was conducted on a monolithically cast steel free deck by Memon et al at the University of Manitoba (Memon, 2003). The 175 mm thick deck spanned between two 9.0 m long girders, spaced at 2.0 m on centre and had 0.5 m cantilevers.

The deck was divided into three equal parts A, B, and C. Segments B and C were steel free decks and segment A was reinforced with steel rebar designed using the empirical design method. Segment B contained #3 Aslan CFRP reinforcement spaced at 200 mm on centre in the transverse direction and 300 mm on centre in the longitudinal direction. Segment C contained #3 Aslan GFRP reinforcement spaced at 150 mm on centre transversely, and 200 mm longitudinally to provide crack control. At segments B and C, 25 mm x 40 mm steel straps were spaced at 1000 mm on centre to provide transverse restraint. Chopped polypropylene fibres were mixed with the concrete at a volume fraction of 0.3%.

Test results showed that the ultimate fatigue resistance of the deck slab was far greater than that required by the predicted maximum lifetime fatigue loading and that the crack widths were related to the magnitude of loads and number of cycles. The maximum crack width under the maximum lifetime fatigue loading was 0.34 mm (Memom, 2003). This research demonstrated the effectiveness of a crack control grid of GFRP reinforcement in steel free decks.

The fatigue resistance of concrete deck slabs reinforced with reduced amounts of steel is higher than deck slabs with increased reinforcement under moving loads (Perdikaris et al, 1988) because fatigue cracks initiate at steel bars. Therefore, having more steel bars increases the number of fatigue cracks and leads to poorer fatigue performance. Research by Memon et al (2003) showed that using GFRP reinforcement significantly increases the fatigue resistance of concrete deck slabs because the Modulus of Elasticity of GFRP is nearly identical to that of concrete and stress risers do not occur at the bars.

GFRP bars are considered an economical alternative to provide crack control in steel free deck slabs. Past concerns have been made about the long-term material performance of the glass fibres exposed to the high alkaline environment in concrete. The stresses in the

bars are very low and the protective matrix surrounding the bars is unlikely to crack. Mufti et al (2005) reported that in a recent durability study which involved extracting concrete cores containing GFRP from in service steel free decks, observed no material defects on the GFRP bars.

2.1.5 *FLANGE SPLITTING STRESSES*

Flange splitting stresses occur in the compression flange of T-beams and can cause longitudinal cracks at the intersection between the flange and the web in unreinforced concrete T-beams (Mufti et al, 1999). A longitudinal crack in this location would lead to premature failure of the beam and requires study for the steel free deck which does not have any transverse reinforcement. Typically, transverse reinforcement is provided in the flange that is adequate to resist this force.

Jaegar and Bakht (2001) have studied this problem and proved the tensile force can be resisted by nominal transverse reinforcement required for deck overhangs at the exterior girders. They have also shown that the steel straps provide adequate confinement for internal panels to resist the splitting force.

Mufti et al, 1999 also studied these effects on the Waterloo Creek Bridge and discovered the following:

- Point loads cause significantly higher tensile stresses compared to distributed loads
- In multi-girder systems, the highest tensile stresses occur at the exterior girder since the slab naturally confines the interior girder. In addition, the addition of steel straps further confines the girders and reduces the magnitude of these forces.

- A finite element analysis also showed that compressive forces exist within a few metres of the ends of the girders and tensile forces exist elsewhere. The tensile forces increase in magnitude towards the middle of the girder span.

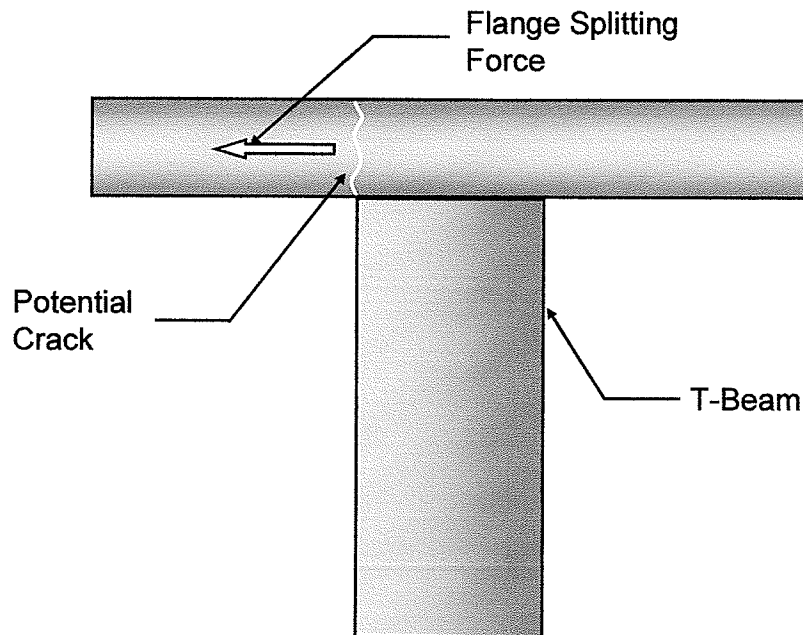


Figure 2-2. Flange Splitting Stresses on T-Beams.

2.1.6 PRECAST STEEL FREE DECK PANELS

The Lindquist Bridge is the only field application of a steel free deck constructed using precast panels. This forestry bridge was constructed in December, 1997 in British Columbia (Mufti et al, 1999). The deck was constructed with full depth precast panels. Circular block-outs at the panel edges over the supporting girders housed clusters of shear connectors that were welded to the girder flanges. The block-outs were grouted after the panels were placed to provide connectivity and composite action between the panels and the supporting girders.

During the initial design and testing of the panels, the transverse free edge was found to be deficient under construction loads when a steel strap was not placed at the edge. A steel strap was placed at the transverse free edge and the area of the straps was halved. This increased the panel capacity to 514 kN, two and one half times the previous failure edge load. The improved capacity was more than adequate to resist the loads from a large back hoe used to drive over the panels prior to grouting them to the girders. Fatigue testing was also performed on the panels by Matsui et al and Mufti et al in 2001. The fatigue tests are discussed in the previous section.

2.1.7 *CURRENT DESIGN STANDARD*

The design guidelines for steel free deck systems are provided in the CHBDC S-00, presently the only bridge design code in the world that incorporates the steel free deck design. Design of the deck is included in section 16.7, FRC (fibre reinforced concrete) Deck Slabs. A detailed list of requirements for fibre reinforced concrete deck slabs is outlined in section 16.7 of the CHBDC. Specific requirements pertinent to the ArchForm system include (CHBDC, 2001):

- Deck slab is composite with parallel supporting beams in the positive moment regions of the beams.
- The spacing of the supporting beams, S , does not exceed 3.0 m.
- The deck slab thickness, t , is at least 175 mm, and not less than $S/15$.
- The height of the haunch between the deck slab and the top of a supporting beam is between 25 and 125 mm, and the projection of the shear connecting devices in the deck slab, t_s , is a minimum of 75 mm; in addition the cover distance between the top of the shear connecting devices and the top surface of the deck slab is a minimum of 75 mm when the slab is not exposed to moisture containing de-icing

chemicals: Otherwise, this cover distance is either a minimum of 100 mm, or the shear connectors are provided with an approved coating.

- The top flanges of the supporting beams are connected with steel straps at a maximum spacing of 1.25 m on centre
- The deck slab is reinforced in negative moment regions at overhangs or over the piers on continuous structures.

The steel straps are designed in accordance with the following formula:

$$A = \frac{F_s S^2 S_f}{Et} \times 10^9 \text{ (mm}^2\text{)} \quad (1)$$

Where F_s is 6.0 for outer panels and 5.0 for inner panels, and E is the modulus of elasticity of the strap material.

The direct or indirect connection of the strap to the supporting beams is designed to have a shear strength in Newtons of at least $200A$. The shear connection devices must be within 200 mm of the nearest strap. The formula is as follows:

$$V_{\text{strap}} > 200A \text{ (N)} \quad (2)$$

The shear resistance of the steel studs is calculated using the following formula:

$$q_{\text{sr}} = 0.5 \phi_{\text{sc}} A_{\text{sc}} (f_c' \times E_c)^{1/2} < \phi_{\text{sc}} F_u A_{\text{sc}} \quad (3)$$

$$\text{where } E_c = 4500 (f_c')^{1/2} \quad (4)$$

2.2 PRECAST DECK PANELS

2.2.1 INTRODUCTION

Bridge owners and agencies continue to seek innovative methods to reduce construction schedules because they recognise the high costs incurred by users of a traffic route due to construction activities. Precast technology is one solution; bridge components are constructed off site and erected on-site in a sequential manner. This approach reduces on-site construction time and the overall construction schedule. The use of precast bridge deck panels is one type of precast construction.

Precast panel technology has been utilised in North America for over forty years (Merril, 2002). The two types of precast panels are partial depth precast panels and full depth precast panels. Partial depth panels are approximately one half of the final bridge deck thickness. The panels are set on top of the girder flanges and cast together with the upper portion of the bridge deck. Connectivity between the panels, girders, and upper portion of the bridge deck is achieved by bond between the concrete surfaces. Full depth panels are the complete final deck thickness. Small full depth block-outs are provided in the panels for shear connectors projecting from the girders (Figure 2-3). After the panels are erected on the girders, the pockets are grouted connecting the bridge deck to the girders. Full composite action between the panels and girders is achieved in both the partial depth and full depth precast concrete panels.

Full depth concrete panel construction requires exposed pour joints at the deck surface which normally necessitate a waterproof membrane to improve their service life. These deck systems require less on-site construction time since only small grout pockets require filling. On the other hand, partial depth panels eliminate all exposed pour joints on the

surface but require additional on-site construction time. There is some concern that reflective cracks initiated at the panel butt joints will propagate to the top of the concrete.

The following will provide a detailed description of partial depth panels which have been investigated as part of this experimental program.

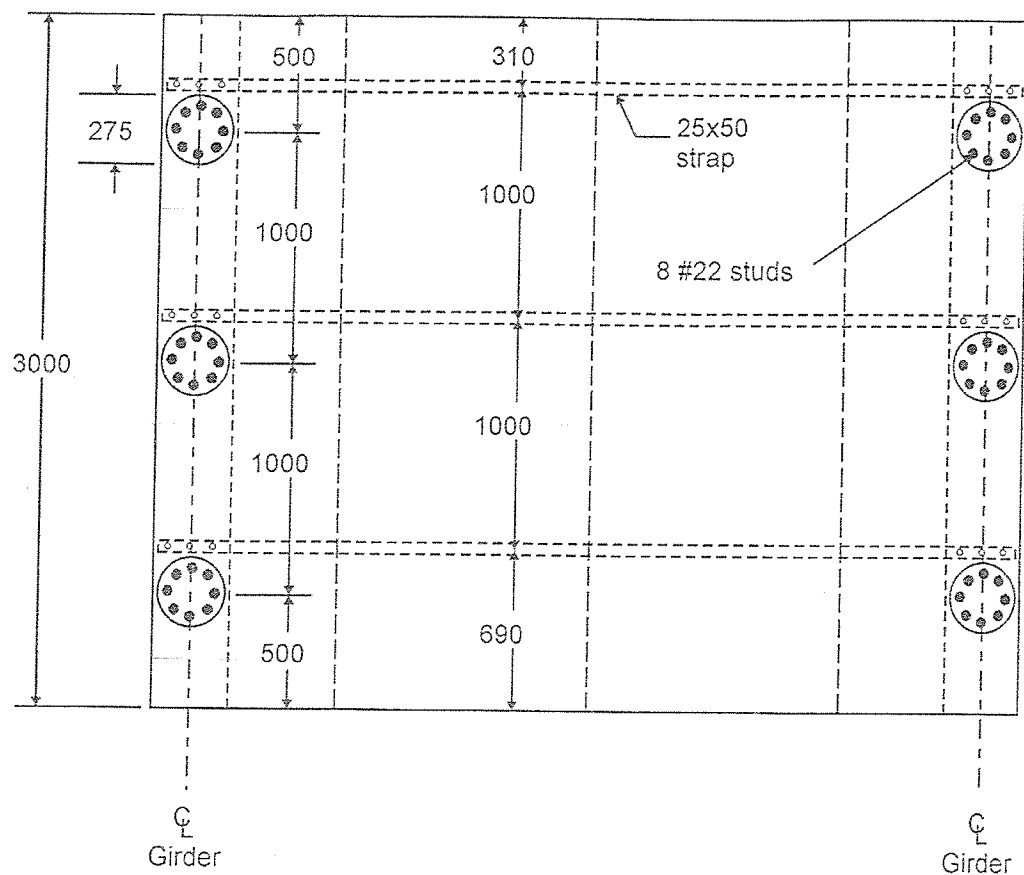


Figure 2-3. Full Depth Precast Panels Block-out Details (Mufiti et al).

2.2.2 *DESIGN DETAILS AND CONSTRUCTION*

REINFORCED OR PRESTRESSED PANELS

Partial depth precast panels are used to reduce on-site construction time and capital construction costs by eliminating formwork and the amount of cast in place concrete. The panels are modular units that span between adjacent girder lines. Typical widths are between 1000 mm to 2500 mm.

Partial depth precast panels are constructed using reinforced concrete or by a combination of reinforced concrete and prestressing. They are cast to fit between girder lines. Each type relies on flexural behaviour to resist loads during construction. In cases where there are multiple girder lines, multiple panels are used. Approximately 75 mm to 100 mm of the panels extend over the edges of the girder flanges. The panels are rectangular and have a constant depth.

Reinforced concrete panels normally contain a single orthogonal grid of reinforcement because the thickness and cover requirements do not permit an upper layer of reinforcing. Hooked bars project from the longitudinal edges and extend over the girder flange to intersect the hooked bars from adjacent panels. The CHDBC requires that these bars be fully developed in order to provide adequate anchorage for arch action in the complete deck slab to occur. However, details developed by the Precast Concrete Institute (PCI) do not require full development of the bars over the girder flanges (PCI, 2002) ensuring that the deck behaves in flexure.

Prestressed panels consist of transverse prestressing strands and transverse and longitudinal reinforcing steel. Reinforcing steel is placed in a single layer above or below the strands or in a double layer that straddles the strands. The reinforcement is placed parallel

to the prestressing strands when a double layer is used. Normally, the prestressing strands are located at the centroid of the panels. The panels are set on top of the girder flanges and the strand ends are either cut off near the panel or extend to lap those of an adjacent panel. PCI details do not require the strand ends or the transverse reinforcement to be developed over the girder flanges resulting in bending behaviour.

JOINTS

Partial depth precast panels have both transverse and longitudinal joints. The transverse butt joints are located where adjacent panels are butted together. These joints can be vertical or bevelled and do not allow cement paste or concrete to flow between them during the pouring of the upper concrete. The end panel terminates at the thickened cast-in-place portion of the bridge that houses the expansion joint. Longitudinal reinforcing steel may project into the cast in place portion of the deck from the precast panel.

Longitudinal joints can be vertical or bevelled. These are wet joints since concrete is poured over their full depth. Reinforcing steel or ends of prestressing strands project through this joint in order to provide continuity between the panels and the girders. As noted above to ensure arching action, the CHBDC requires full development of the reinforcing steel ends.

The upper surface of the panels is typically finished with a broom or raked finish with minimum amplitude. Adequate bond between the precast concrete and cast in place concrete is required to transfer horizontal shear forces to ensure that the slab acts monolithically. Research performed by the Texas DOT proved that only a broom finish is required to transfer the small horizontal shear forces across this joint. This is presently their design standard. Conversely, the CHBDC (2001) requires that a raked finish be provided with a minimum amplitude of 2 mm.

PANEL BEARING DETAIL

The panel bearing detail is the most critical construction detail for partial depth precast. At this location, vehicle or truck live loads are transferred from the panel to the girders. During placement of the cast-in-place concrete, mortar must be allowed to flow between the girder flange and underside of the panels. This provides a mechanism to transfer the vertical loads to the girders by bearing rather than relying on shear friction between concrete surfaces.

Precast panels are placed on a temporary supporting material during installation. Asphalt impregnated fibreboard or high density extruded polystyrene foam are the most common materials used to temporarily support the panels. These materials are used to eliminate irregularities in the girder flange and to account for girder camber. Experience has shown that fibre bolster board creeps, affecting the gap between the girder and underside of the panels when the panels are set too far in advance of pouring of cast-in-place concrete (Merril, 2002). Polystyrene can be cut to match the camber of the girders to avoid this problem.

Premature failures of bridge decks constructed using partial depth panels have occurred in the State of Florida and most recently on the I-81 bridge structure in Washington State. In the 1980s the State of Florida experimented with the construction of bridge decks using partial depth panels. Interviews with a Florida DOT engineer revealed that these bridge decks performed extremely poorly. Large cracks occurred at the longitudinal joints and extensive deterioration of concrete and spalling was observed. Complete rehabilitation of the deck was required in some locations. The premature deterioration that occurred was likely caused by a poor bearing detail.

A report was recently published by the Virginia Transportation Research Council which describes problems associated with the bridge deck on the I-81 Bridge over the New River in Radford, constructed in 1985 (Brown et al, 2003). The bridge consists of 10 continuous spans totalling approximately 505 m in length. Each span is comprised of five haunched steel I-girders spaced at 2.74 m on centre. Figure 2-4 shows the as-built bearing detail for this structure. This route carries an extremely high volume of interstate truck traffic.

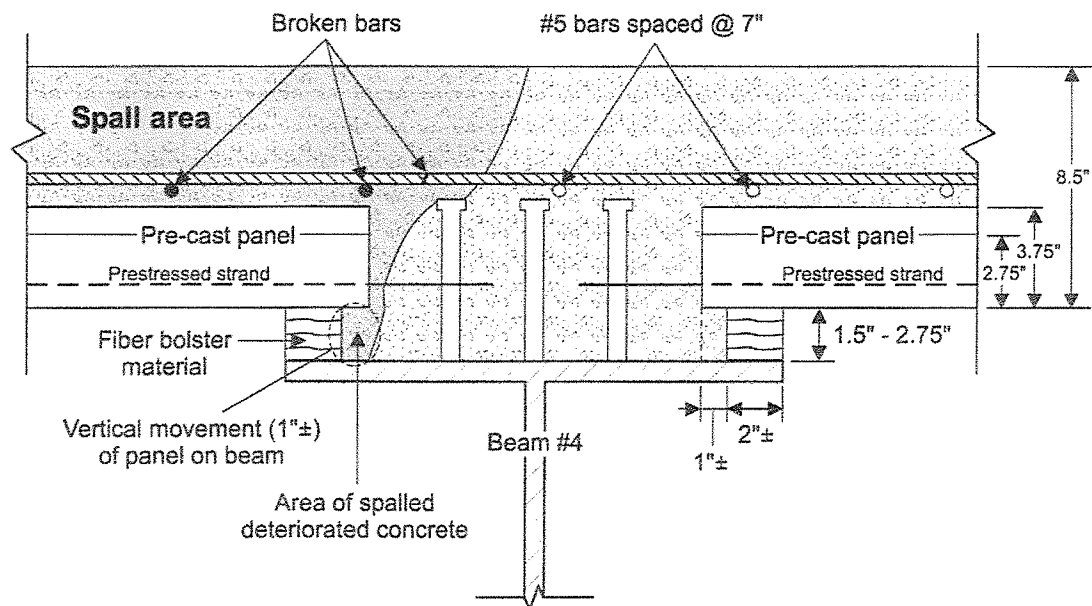


Figure 2-4. As-built Panel Bearing Detail for the I-81 Bridge over the New River (Brown et al , 2003).

The bridge deck was constructed with 95 mm thick prestressed panels complete with a 120 mm thick cast in place wearing surface. Progressive deterioration of the deck concrete has occurred to the depth of the prestressing strands in the worst areas since construction was completed. Frequent patching of the concrete has been required. Deterioration has occurred at the longitudinal joints above the girders, resulting in the partial loss of composite action between the girders and deck slab. The report indicates that where the edges of the

panels bear on the fibreboard and cast-in-place (CIP) concrete, the panels do not extend beyond the fibreboard a sufficient distance to provide permanent support. The report recommends that the fibreboard should be replaced with a more supportive material like steel shims, concrete, shot Crete, or dry-packed cement mortar.

CONSTRUCTION

Proper execution and special attention to details is critical to the construction of any bridge deck, including those constructed with precast panels. To better understand the design requirements and field performance, the construction method must be clearly understood. Once the panels are cast and cured they are delivered to the site and placed on top of the girders. This is typically performed with a small crane that lifts the panels using hooks cast into the concrete. The construction sequence of the precast bridge deck system is as follows:

1. Cast and cure the precast deck panels off site.
2. Survey the top of the girder flanges and determine camber adjustments.
3. Cut and glue in place polystyrene supports on top of girder flanges.
4. Deliver the panels to the site.
5. Erect the precast deck panels onto the bridge girders.
6. Verify that the minimum required bearing width has been met and that there is a 12 mm wide gap at a minimum of 1200 mm along the length of the polystyrene supports to allow air to escape during concrete placement.
7. Install the formwork for deck overhangs and at expansion joint dams.
8. Place the upper deck lift and barrier reinforcing steel.
9. Pour and cure cast-in-place upper lift concrete, ensuring that the top of precast panels are saturated surface dry prior to concrete placement.
10. Remove formwork.

At first glance it appears that the construction of the deck using precast panels involves more work than conventional cast-in-place construction. This is not the case for two main reasons; Very little forming is required and the quantity of cast-in-place concrete is nearly halved. Savings resulting from the elimination of formwork and savings to project schedule are estimated to be as much as \$80/m² of bridge deck.

2.2.3 *HISTORY AND FIELD PERFORMANCE*

In North America, the Texas Department of Transportation (TxDOT) has the most experience with partial depth precast panels. State research on partial depth precast panels began in 1963, and a limited number were constructed from that time until the early 1980s (Merril, 2002). Research and field performance proved that this method of constructing a bridge deck was feasible and did not significantly alter the short time or long term (fatigue) performance of the bridge decks (Merril, 2002).

Over 85% of all newly constructed bridge decks in Texas use precast panels. There are well over a thousand bridges in the State of Texas that have been constructed with partial depth precast panels. These panels are the preferred alternative since they are much more economical to construct and require similar maintenance as a monolithic reinforced concrete deck (Merril, 2002). In fact, testing has shown that they have superior fatigue performance resistance when compared to monolithically cast reinforced concrete decks.

Two areas of concern that have been noted are the panel joints and the panel bearing detail. At both the longitudinal and transverse joints, reflective cracking often occurs (Merril, 2004). These cracks are hairline type cracks which do not cause maintenance problems. Texas and North Carolina have constructed well over a thousand bridge decks using

precast panels. Interviews with state Department of Transportation engineers revealed that although these types of cracks do occur from time to time, their widths are minimal and do not affect the service or structural performance of the decks (Merril, 2004). Bridge decks in the state of Texas are in a relatively benign corrosion environment, and the bridge decks in North Carolina are exposed to sea salt spray.

Joints have been problematic in other areas, namely the State of Florida and Virginia State. This may be related to the panel bearing detail. As previously noted, the State of Florida has discontinued using the panels due to several maintenance problems with bridges constructed using this technology in the 1980's. The I-81 Bridge over the New River in Radford, Virginia has also experienced several maintenance problems to the extent that the composite action between the bridge deck and girder was compromised (Brown et al, 2003). It has been suggested that the end bearing detail was inadequate since the panels are resting on fibreboards, not mortar.

The horizontal joint between the panels and cast-in-place concrete is treated with a broom finish. The joint is moistened with water to a saturated surface dry condition prior to placement of the upper lift concrete. No delamination failures have been observed at this joint in the State of Texas (Merril, 2004).

2.2.4 *FATIGUE PERFORMANCE*

Static loading is typically not the governing design criteria for bridge decks. Dynamic loading from heavy trucks is the critical design load and will bring to light any design or construction defects that lead to increased maintenance.

The TxDOT carried out extensive fatigue performance research prior to implementing partial depth precast panels. Their research showed that the prestressed panels performed better than monolithically cast in place bridge decks. This can be attributed to prestressing that ensures that the concrete does not crack. In addition, a higher strength concrete was used for these slabs. The slab tested was compared to a cast-in-place deck slab reinforced with orthogonal top and bottom grids of rebar designed using the Ontario Design Method (0.2 % orthogonal reinforcement). Both decks performed satisfactorily under 5 million cycles of loading ranging between 22.2 kN and 116 kN. The deck section constructed using partial depth prestressed precast panels proved to be stiffer and stronger during fatigue testing.

2.2.5 *BRIDGE CODE REVIEW*

Partial depth precast panels must be designed to resist construction loading during the placement of the upper lift concrete in addition to loads imposed on them during transportation and erection. Construction loading includes the self weight of the panels, weight of wet concrete on the panels and the weight of equipment and workers during the placement of the upper lift concrete.

The most significant loading occurs in the event that the concrete is poured using buggies rather than by a concrete pump. In order to understand these loads a review of the CHBDC, CSA and AASHTO design codes was carried out to determine the loading criteria for formwork and falsework. Table 2-1 provides a summary:

Table 2-1. Summary of Loading on Partial Depth Precast Panels.

Design Code	Design Load (Live Load Unless Otherwise Noted)	Live Load/Total Load Factor	Dead Load Factor
CHBDC 2001	0.5 kPa (min)	1.445 (Live Load)	1.25
AASHTO LRFD 2004	0.5 kPa (min)	1.5 (Live Load)	1.25
AASHTO* Falsework Design	0.96 kPa (min) - Increase to 2.16 kPa motorized buggies are used to place concrete - Total live load + dead load shall be > 4.8 kPa	Total factor of safety = 2.5	N/A
AASHTO* Formwork Design	2.4 kPa (min) - Increase to 3.6 kPa motorized buggies are used to place concrete - Total live load + dead load shall be > 4.8 kPa or 6.0 kPa when motorized buggies are used to place concrete	Total factor of safety = 2.5	N/A
CAN/CSA-S269	1.2 kPa + 3.6 kN point load	Total factor of safety between 2 and 3	N/A

* AASHTO Guide Design Specifications for Bridge Temporary Works, 1995

Based on this information, the CHBDC and the AASHTO LRFD 2004 allows the designer the most flexibility to design for realistic loads during construction. Careful consideration and calculation of the loads is required since there is not a large factor of safety. When concrete buggies are not used, and concrete is placed using a concrete pump, the loading criteria by AASHTO Design Specifications for Bridge Temporary works and the CAN/CSA-S269 may make it more difficult to provide ingenious designs.

3.0 DESIGN OF ARCHFORM PANELS

3.1 INTRODUCTION

The purpose of this section of the report is to describe the design of the ArchForm system including the panels and deck. The design of the ArchForm system included the design of the panels to support temporary construction loads and the design of the permanent deck system to support truck wheel loading. Both the numerical design and the detailing are discussed.

The design philosophy for the panels and the completed deck were completely different. The panels were assumed to behave as bending elements since no lateral or longitudinal confinement would be provided during construction. The complete deck system was designed to support truck loading assuming that the behaviour would be by arching action and that the failure mode would be punching shear.

3.2 DESIGN AND DETAILING

PANEL DESIGN

The panels were designed to support their self weight, the weight of wet concrete during the placement of the upper lift of deck concrete, and the weight of any workers or equipment that may be applied to the panels during construction. In addition, the design needed to be robust enough to withstand handling loads. The design was performed in accordance with the CHBDC. Since the span to depth ratio of the panels is quite large and no lateral or

longitudinal constraint would be provided during construction, the panels would behave in positive bending.

Both haunch shaped and a rectangular shaped panels were examined during the design. The haunched shape was selected since it provides separation from the steel strap and the shape will improve the shear capacity of the deck system under wheel loading. Research by Newhook (1997) showed that total deck thickness was expected to contribute to the capacity provided that a straight line from the load point passed through the concrete haunch (Figure 3-1). Therefore the width of the haunches was selected based on this parameter.

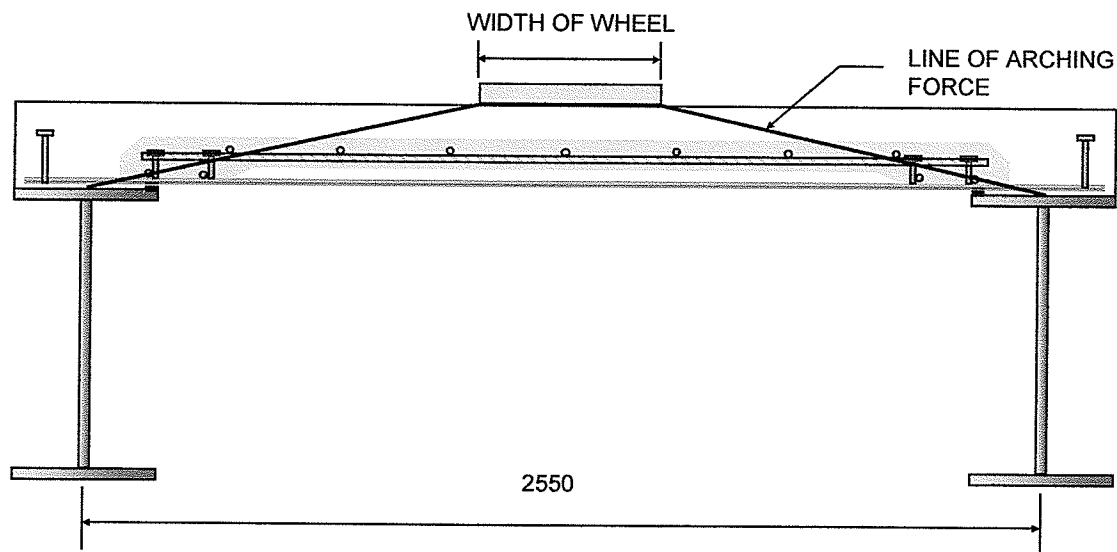


Figure 3-1. Theoretical Punching Shear Crack.

Presently the minimum thickness for a steel free deck is 175 mm. This was selected as the thickness of the complete deck system. The complete deck system was designed in accordance with the CHBDC to ensure that both the concrete thickness was adequate and that a readily available steel strap size was achievable.

An initial panel thickness of 75 mm was selected for the panels in order to provide adequate cover for the GFRP reinforcement grid and to reduce the weight of the panels. This was also viewed as the least conservative thickness. Based on this geometry, both the internal reinforcement and the external steel strap would provide flexural restraint for the panel. However, because the steel strap is not fully bonded to the concrete, it is difficult to predict the actual flexural capacity of the panel.

A nominal grid of reinforcement is used in second generation steel free deck to provide crack control and prevent the formation of full depth cracks. This reinforcement was incorporated into the panel design to provide flexural resistance for the panels and crack control for the completed deck system. A grid of #3 or 9.5 mm diameter GFRP bars at 300 mm on centre in both directions was used. In the complete deck system, the steel strap will provide all of the transverse restraint.

As a precaution, one of the three panels tested was constructed with the transverse GFRP bars spaced at 150 mm on centre. This was done to provide additional flexural resistance for the slab under construction loading.

3.2.1 *GEOMETRIC DESIGN*

The design of each aspect of the steel free deck system used in this experimental program is discussed below. Detail assembly drawings are found in Appendix B.

3.2.2 STRAP SIZE

Strap spacing between 1.0 m and 1.2 m are the most efficient spacing for steel free decks (Newhook, 1997). A strap spacing of 1.2 m was selected for the design. The sizes of the steel strap based on the geometric properties of the deck were calculated as follows:

$$A = (F_s \times S^2 \times S_i) / (E \times t)$$

$$A = (5.0 \times 2,550^2 \times 1,200) / (200,000 \times 175) = 1,115 \text{ mm}^2$$

Assuming that the strap was 25 mm thick yields a strap width of 44.6 mm. A steel strap measuring 25 mm x 50 mm was selected.

3.2.3 REINFORCEMENT DESIGN

Based on traditional flexural design principles the capacity of the panels could not be easily calculated since there was no strain compatibility between the steel straps and the concrete. As noted, the capacity of panels was predicted to be somewhere between the flexural capacity of the GFRP reinforcement (lower bound solution) and the flexural capacity of the steel straps (upper bound solution) assuming that the steel straps were bonded to the deck concrete.

The GFRP was designed using the ISIS Manual. The flexural capacity of the panels was calculated to be 3.7 kN-m/m assuming that $f_c' = 35 \text{ MPa}$ and the #3 GFRP bars were spaced at 300 mm on centre. The flexural capacity of the panels assuming that the steel straps acted as continuously bonded reinforcement was 32.6 kN-m/m. Additional capacity from the GFRP bars was neglected in this calculation since the GFRP was very close to the neutral axis of the section.

The factored design moment of the panels was 6.9 kN-m/m. The dead load comprised of 91% of this moment based on the CHBDC live load of 0.5 kPa.

A grid of 10 M or #3 GFRP bars was selected recognising that this spacing would provide adequate crack control for the completed deck system. As a contingency, the third panel was constructed by providing the same transverse bars at 150 mm on centre as opposed to 300 mm on centre.

3.2.4 *SHEAR CONNECTOR DESIGN*

The design of the shear connectors was based on the formula in the CHBDC. Equations two to four were used in the analysis. The shear connectors were required to resist a shear force of 223 kN as shown below.

$$V = 200 \times A / 1000 \quad (2)$$

$$V = 200 \times 1,115 \text{ mm}^2 / 1000 = 223 \text{ kN}$$

Shear connectors measuring 22 mm diameter were utilised. The capacity of the shear connectors is equal to the lesser of the steel or concrete capacity. Calculations were:

$$E_c = 4500 (35)^{1/2} = 26,622 \text{ MPa} \quad (4)$$

$$q_{sr} = 0.5 \phi_{sc} A_{sc} (f'_c \times E_c)^{1/2} < \phi_{sc} F_u A_{sc} \quad (3)$$

$$q_{sr} = 0.5 \times 0.85 \times 380 \times (35 \times 26,622)^{1/2} / 1000 = 156 \text{ kN}$$

$$< 0.85 \times 410 \times 380 / 1000 = 132 \text{ kN}$$

Therefore $q_{sr} = 132 \text{ kN}$

The total number of shear connectors required = $223 \text{ kN} / 132 \text{ kN} = 1.69$ for the deck design. Analysis for the panel design indicated that two 19 mm diameter shear connectors were required to ensure that the steel strap would yield prior to failure of the shear connectors. Only one of the shear connectors within the panel was located over the girder flange and its capacity was considered for the design of the deck system.

Two 22 mm diameter by 75 mm long Nelson studs were located within the precast panel and a third 22 mm diameter by 150 mm long Nelson stud was located in the deck. This third stud was positioned furthest to the outside of the panel as shown in Figure 3-2.

3.2.5 *CHAMFER DETAILS*

In phase 1 testing only the transverse butt joints were chamfered. Both the transverse and longitudinal joints were chamfered on the panels constructed for the complete deck system (phase 2). The purpose of the chamfer was to increase the surface area of this joint and to eliminate all right angle corners. This detail is shown in Figure 3-2.

3.2.6 *BEARING DETAILS*

The bearing detail for the panels resting on the top flange of the girders is critical. The detail is similar to that presently used by TxDOT and PCI. This detail allowed the permanent grout pad to be at least 12 mm in height x 40 mm in width (Figure 3-2).

3.2.7 *END DETAIL*

The CHBDC describes four different end details which ensure that there is adequate longitudinal restraint to develop arching action when a wheel load is applied at the end of bridge. The channel detail was selected for ease of construction and the deck concrete was

increased to 250 mm in accordance with this detail. This small portion of the deck was cast in place.

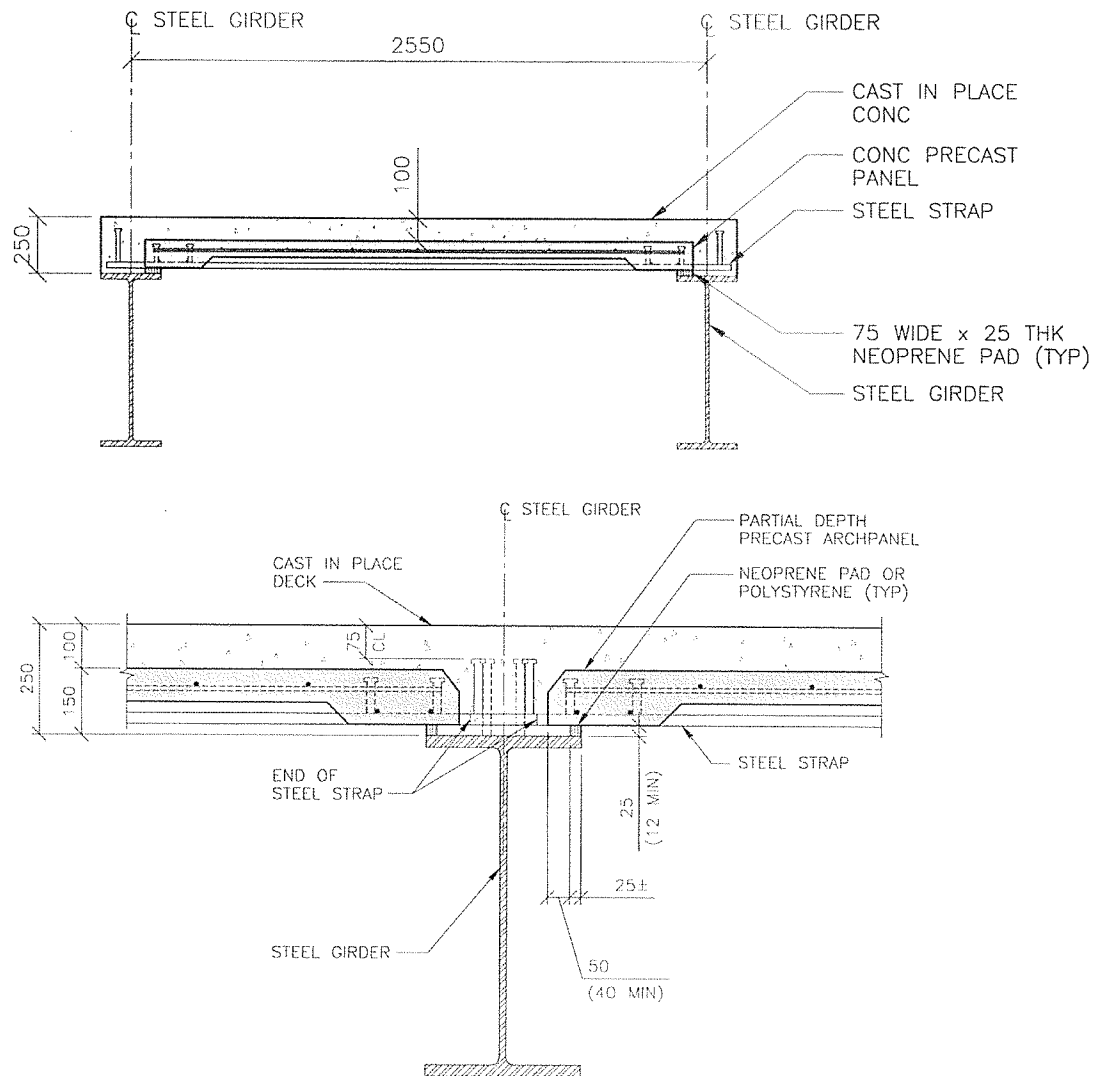


Figure 3-2. Archform Cross Section and Details.

3.3 MATERIAL SELECTION

For all of the test specimens a standard 35 MPa concrete mix was utilised. The mix proportions used for the concrete in the phase 1 testing and phase 2 testing are summarised in table 3-1.

Table 3-1. Concrete Mix Proportions.

Property	Phase 1 Panel Mix	Phase 2 Panel Mix	Phase 2 Deck Mix
Specified Air	4-6%	4-6%	4-7%
Specified Slump	140 +/- 20mm	140 +/- 20mm	100 +/- 25mm
Specified Maximum Aggregate Size (mm)	19	19	19
Sand (kg/m ³)	851	850	800
Aggregate (kg/m ³)	1090	1109	1060
Cement (kg/m ³)	332	329	332
Water (kg/m ³)	117	79	145
Water/Cement Ratio	0.35	0.24	0.44
Air Entrainment (ml/m ³)	15	15	15
Plasticizer (ml/m ³)	850	805	834

This mix was used since it is a relatively uncomplicated concrete mix that can be easily achieved by any ready-mix concrete plant. The steel straps conformed to CAN/CSA G40.21, 300W steel and the Nelson studs were constructed in accordance with ASTM A-108 with a yield strength of 345 MPa. Aslan 100 GFRP bars were used as panel reinforcement. The guaranteed tensile strength of the bars was 760 MPa and the modulus of elasticity was 40,800 MPa. The corresponding rupture strain was 18,627 μ strain compared with 2,000 μ strain for steel. All bars were nine millimetres in diameter and were supplied by Hughes Brothers.

4.0 EXPERIMENTAL INVESTIGATION

4.1 INTRODUCTION

This section of the report describes the experimental design for both the testing of the panels and the complete deck system which are referred to as phase one and phase two. In phase one, a total of three panel specimens were constructed and tested. Two panels were tested under both static parallel line loads to simulate uniform loading and one panel was tested under a point load at either ends of the panel. Once this was complete, another three panels were constructed, erected on the steel girders in the laboratory and a concrete overlay was cast to form the deck system. Three static wheel load tests were conducted on this deck specimen.

The purpose of phase one testing was to prove that the panels could safely resist construction loading which included their own self weight and construction loading during the placement of wet concrete. In addition, the panels were handled in a manner representative of typical construction conditions to ensure that they could not be easily damaged during transportation and erection. The purpose of the phase two testing was to prove that the deck would behave the same way as a cast in place steel free deck.

4.2 EXPERIMENTAL PHASE 1 – PANELS ONLY

4.2.1 DESCRIPTION OF EXPERIMENTAL MODEL

PANEL DETAILS

Three panel specimens were constructed and tested. Panels one and three were tested by applying two parallel line loads applied at the 1/3 points of the panels (Figure 4-1). This was used to simulate uniform loading on the panels which would occur when wet concrete was placed. Panel two was tested under a point load at the mid span of both of the transverse free edges. This load was used to simulate a large amount of concrete placed in one location and was performed at the free edge since this is the most critical loading location.

All of the panels were constructed with identical cross sections and plan areas as shown in Figures 4-1. The plan dimension of all panels was 2,425 mm wide x 3,600 mm long. The panel width was dictated by the centre to centre spacing of the support beams. The panel thickness varied from 125 mm at the ends to 75 mm at mid span. Three 25 mm x 50 mm steel straps were cast in each of the panels. The straps were located 600 mm from each of the ends and evenly spaced at 1,200 mm on centre.

Nelson studs were welded to the ends of the straps that were embedded in the concrete. The studs were 19 mm in diameter and 75 mm long. All three panels contained an orthogonal grid of #3 or 10M GFRP reinforcement. The transverse layer was placed on the bottom and 25 mm of cover was provided at the mid span. The longitudinal layer was placed on top of the transverse layer. Panel numbers one and two contained a grid of reinforcement at 300 mm on centre. In panel number three which was tested under the parallel line loads, the spacing of the transverse reinforcement was tightened to 150 mm on

centre. An additional two longitudinal bars were added at the bottom of the haunch to provide flexural strength during the handling of the panels.

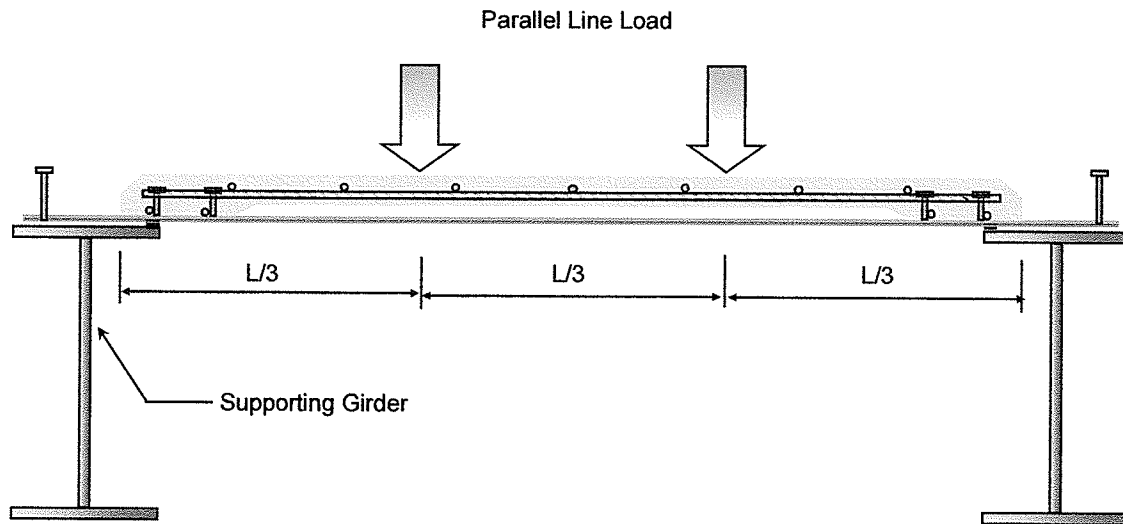


Figure 4-1. Distributed Load Test Set-up.

PANEL CONSTRUCTION AND ERECTION

A total of three panels were designed, constructed and tested for both the panel testing and deck testing. All of the panels were constructed at Lafarge's precast plant located in the City of Winnipeg. The panels were constructed on a level wood bench with their top side up. Styrofoam was used to form the haunch and 38 x 89 mm lumber was cut to form the chamfer at the haunch. Polyethylene sheets were used to cover the Styrofoam to prevent it from absorbing water during placement of the concrete (Figure 4-2).

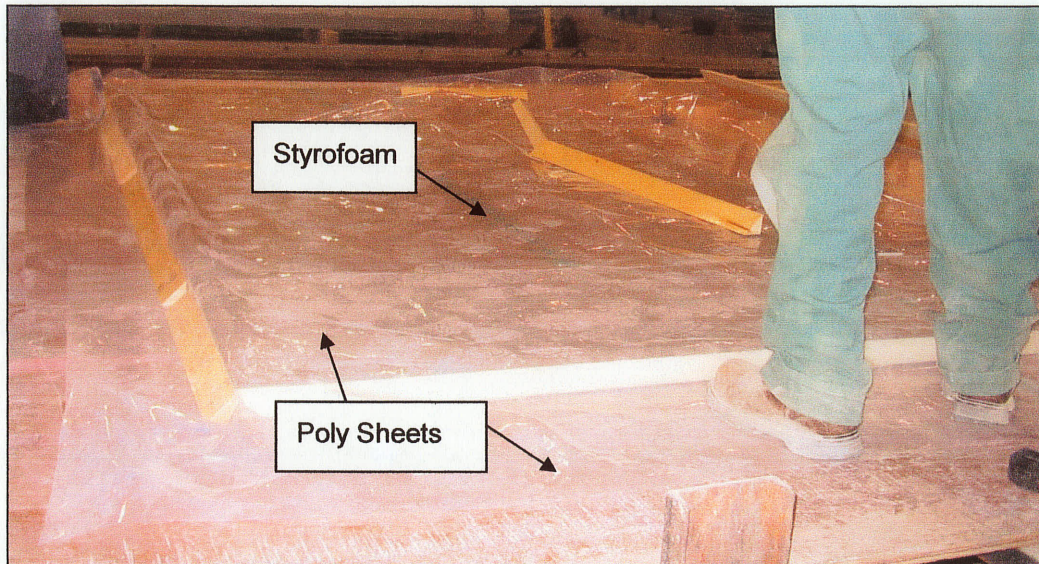


Figure 4-2. Formwork for Concrete Panels.

Nelson studs were welded to the straps and the straps were placed on the wood bench where the Styrofoam had been notched. A single strip of Styrofoam was placed over the steel straps between the panel haunches. Wood blocking was installed to secure the steel straps in place and 19 mm thick plywood was used to form the perimeter of the panels as shown in Figure 4-3.

Concrete was mixed and poured at the precast plant as shown in Figure 4-4. Concrete was placed using a concrete bucket; shovels and rakes were used to provide a rough levelling. A vibrating screed was used to level the concrete and provide compaction and no handheld vibrators were required since the concrete depth was a maximum of 125 mm. The panels received a smooth trowel finish for the phase one testing.

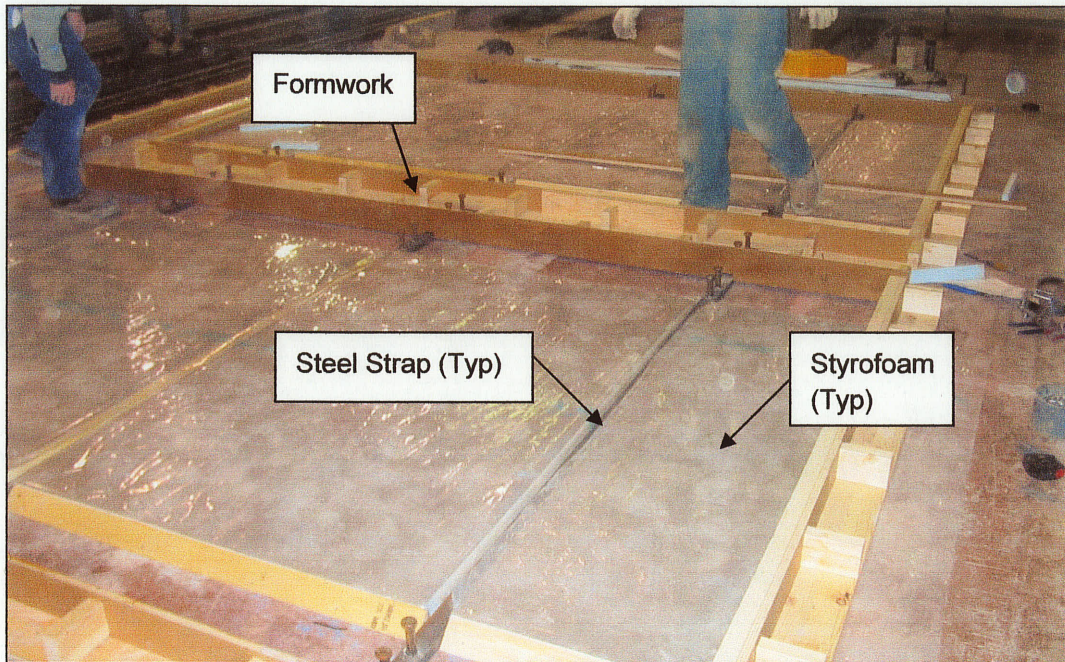


Figure 4-3. Formwork for the Precast Panels.

Once the concrete had cured and reached a compressive strength of 30 MPa determined by compressive concrete strength tests, the panels were lifted onto a flatbed semi-trailer and transported to the University of Manitoba Structural Engineering Laboratory for testing.

During the moving and transportation of the panels, special attention was paid to ensure that these panels were not handled any differently than panels constructed for industry use. This provided added insurance that they were rugged enough to endure construction handling loads.



Figure 4-4. Compaction of Concrete using Vibratory Concrete Screed.

PRECAST PANEL ERECTION

The precast panels were lifted using an additional four Nelson studs at each of the four panel corners or by wrapping a strap underneath the steel strap at each of the four corners (Figure 4-5). These pick-up points were used to move the panels from their place of construction onto a flat bed trailer and from the trailer onto the tops of the girders. Prior to placing the panels on the steel girders, neoprene pads were set in place on top of the girder flange. Minor adjustments of these pads occurred once the panels were placed on the girders.



Figure 4-5. Handling and Moving of Panels.

4.2.2 *INSTRUMENTATION OF SPECIMENS*

Instrumentation of the panels was required in order to measure the strain in the steel straps and the strain in the GFRP reinforcement. Different instrumentation was used in panels one and two since they were tested differently. The instrumentation on panel number three contained all gauges used in panels one and two in case it was needed as a back up for any of the tests.

DISTRIBUTED LOAD TEST ONE AND TWO

In the distributed load test one and two, both the strain in the steel straps and transverse reinforcement was measured to determine how the load was shared under bending. As previously discussed, it was unclear how the panels would behave under bending and what proportion of tensile load would be resisted by the steel straps compared with the GFRP bars cast in the concrete. The instrumentation is shown in Figures 4-6 and 4-7. Three steel straps were instrumented with strain gauges as well as three transverse GFRP bars which were located directly over the steel straps.

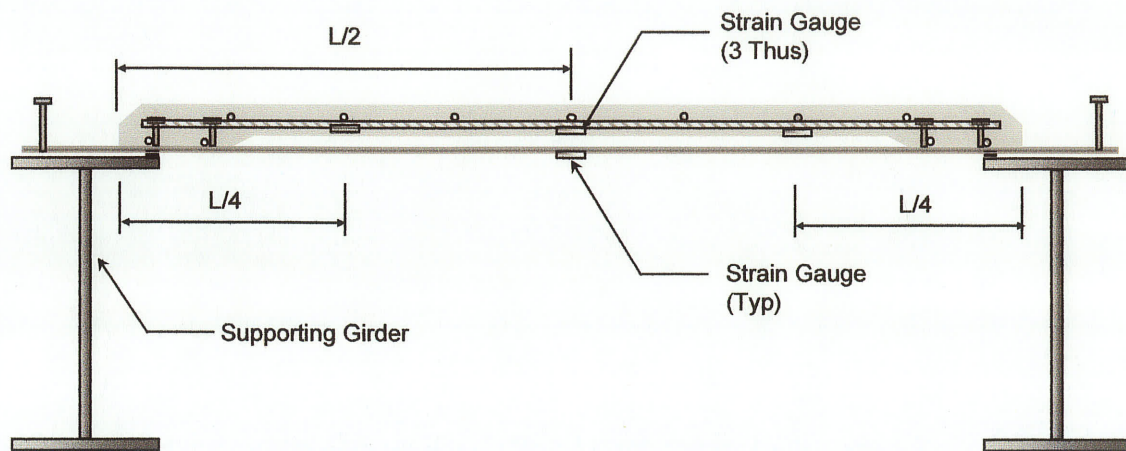


Figure 4-6. Location of Strain Gauges on GFRP Reinforcement and Steel Straps.

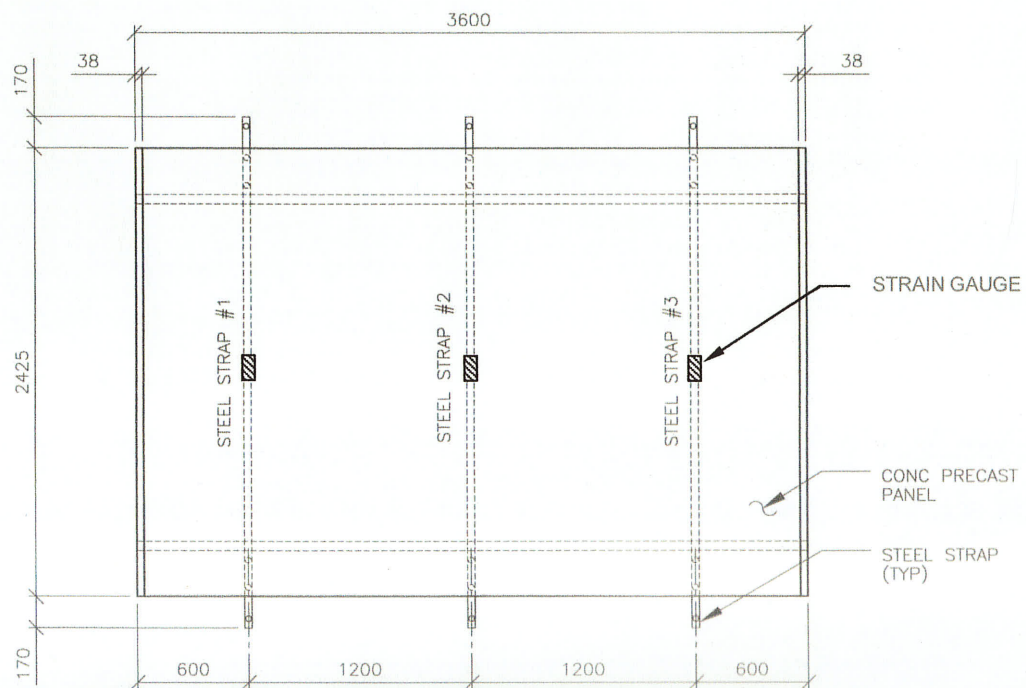


Figure 4-7. Location of Strain Gauges on Steel Straps.

Linear variable displacement transducers (LVDTs) were used to measure the mid span deflection of the panels at three points along the panel's length. Three LVDTs were placed directly above each of the steel straps as shown in Figure 4-8. Supports for these LVDTs were set on top of the girders and girder deflection was not measured.

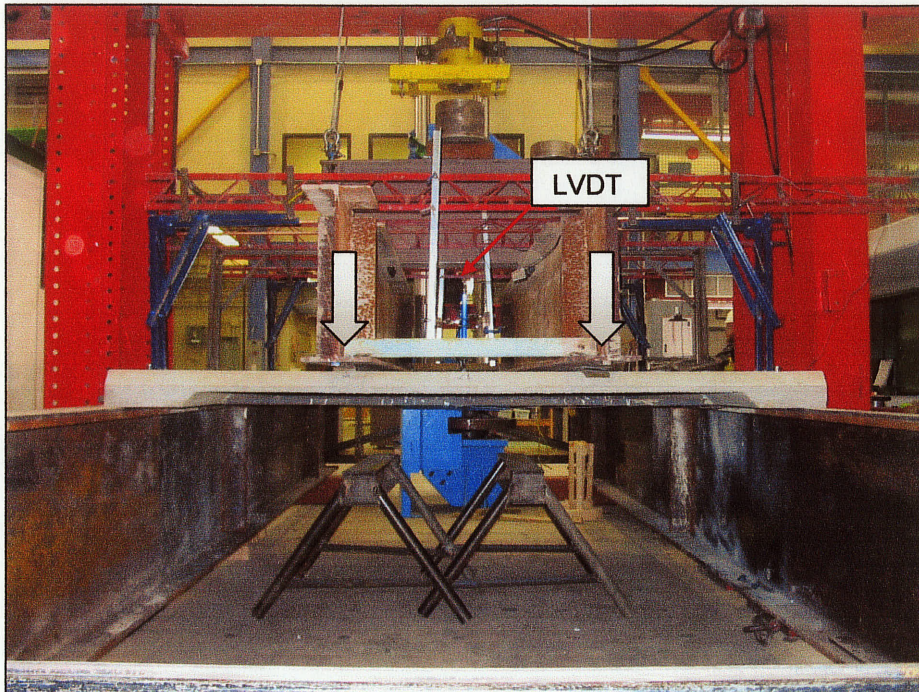


Figure 4-8. Location of LVDT for Panel Distributed Load Test.

POINT LOAD TEST ONE AND TWO

The purpose of the point load tests was to determine the capacity of the panels and behaviour under a point load. Load would either be transferred directly to the edges of the panels or to the first steel strap and then to the edges of the panels or a combination of the above. In the latter case, the steel strap would need to attract the majority of the load. Two transverse bars were instrumented with three strain gauges located at the middle of the bar and at quarter points. These bars were located between the panel's transverse free edge and the edge steel strap and directly above the edge steel strap. A total of five longitudinal

bars were instrumented with strain gauges. These gauges were located at the point of the bar directly above the end steel strap as shown in Figure 4-9 below.

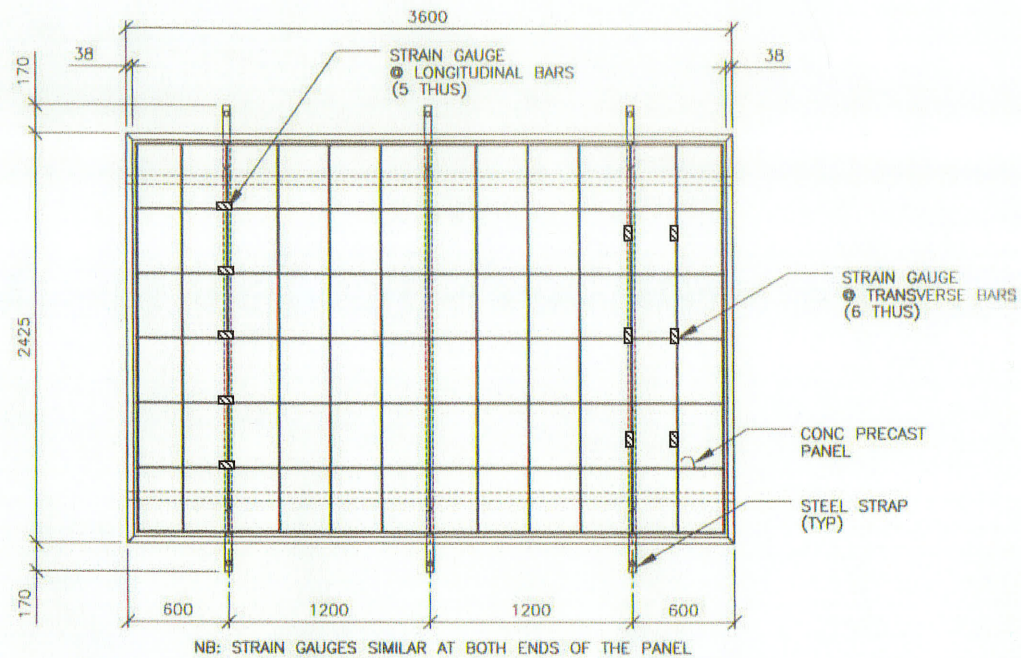


Figure 4-9. Strain Gauge Locations on Transverse and Longitudinal GFRP Bars.

Two LVDTs were used to measure the mid span deflection of the panel. One transducer was located at the transverse free edge adjacent the point load and the other was located on the opposite side of the point load as shown in Figure 4-10. Table 4-1 provides a summary of the strain gauges used for each of the tests.

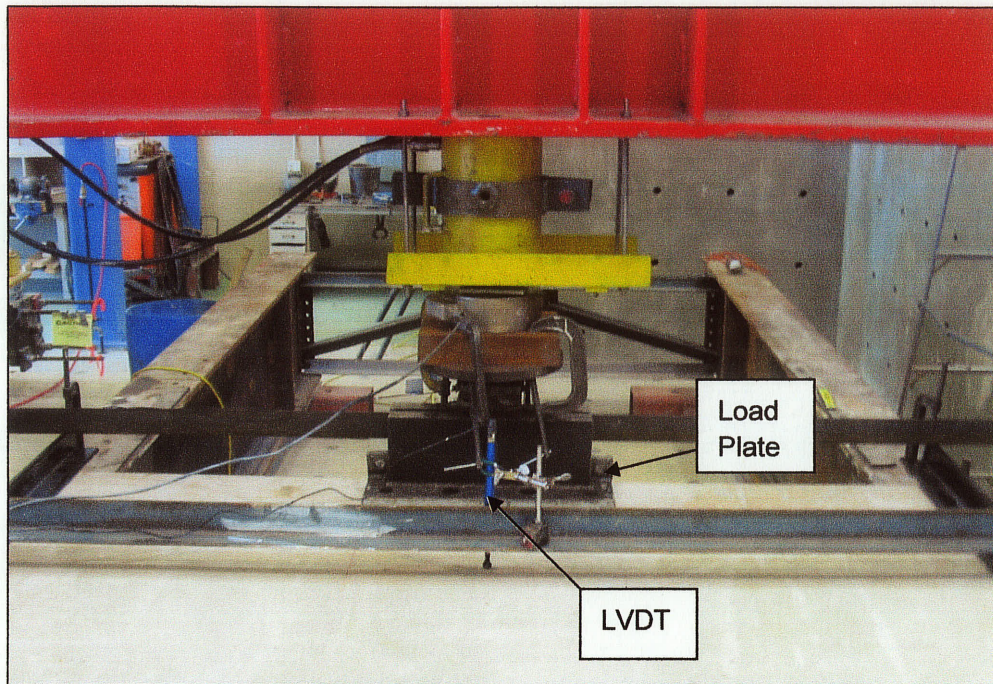


Figure 4-10. Location of LVDT on Inside Face of Load Plate.

Table 4-1. Summary of Instrumentation on Each Panel.

Panel #	Strain Measurement			Deflection Measurement
	Steel Straps	Transverse GFRP	Longitudinal GFRP	LVDT
1 (Distributed Load Test)	Midspan on bottom of all three steel straps (Total of three gauges)	- Placed on the three GFRP bars located directly above each steel strap - Gauges at midspan and $\frac{1}{4}$ points - Note that middle bar had a gauge at midspan and at only on one side at the quarter point (Total of eight gauges)	None	Placed on top of the slab at the midspan of each of the steel straps (Total of three)
2 (Point Load Test)	Midspan on bottom of all three steel straps	- Placed on two GFRP bars located directly above each of the outside steel straps - Gauges at midspan and $\frac{1}{4}$ points (Total of six gauges)	- Placed on the five middle GFRP bars - Gauges located at either ends of each of the bars	Placed on top of the slab at the midspan on either side of the test plate (Total of two)
3 (Distributed Load Test)	Instrumented with all strain gauges on both panel one and panel two. LVDTs same as test number one.			

4.2.3 TESTING SETUP

A test specimen was constructed using two parallel girders measuring approximately 9,100 mm in length and spaced at 2,550 mm on centre at the University of Manitoba Structures Laboratory. The clear distance between the girder flanges was 2,325 mm which provided a continuous bearing width of 75 mm for the panels. Cross bracing was provided at the ends of the test specimen as shown in Figure 4-11. The girders were W760 and 76 mm x 76 mm x 12 mm angles were used to construct the cross bracing. The girders were set on HSS blocking and all tests were conducted at the mid span of the girders.

A load frame constructed of four steel columns and a steel beam spanning between the columns was used to support the hydraulic jack that applied load during the testing. This is shown in Figure 4-11 below. High strength bars were used to secure the load frame to the floor.

Two W530 x 33 girders measuring 3500 mm in length were used to simulate line loading. A transverse beam transferred load from the hydraulic jack to the parallel girders as shown in Figure 4-11. In order to ensure that the load was applied at 1/3 points, a 6 mm deep x 12 mm wide steel bar was welded to the underside of the girders at the centreline of the web. These bars were placed on top of neoprene pads.

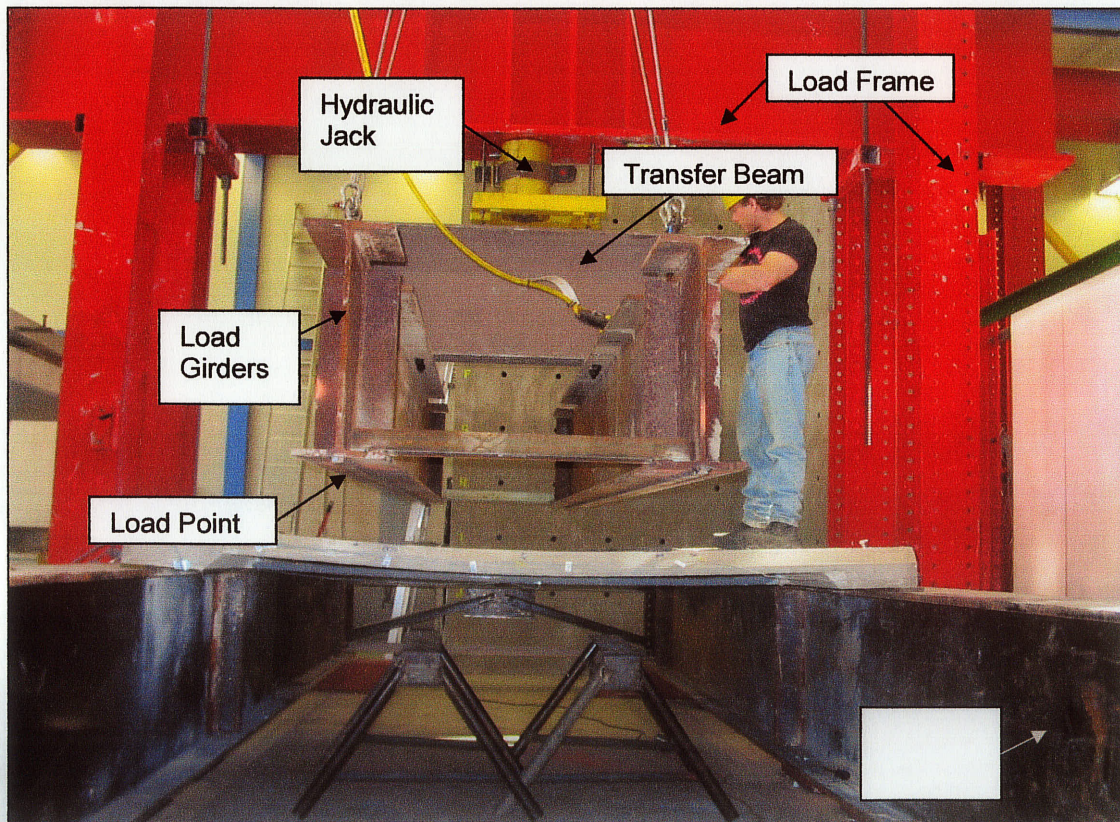


Figure 4-11. Test Set-up for Distributed Load Test.

During the point load tests, a stiffened steel plate measuring 305 mm x 610 mm in plan was used to simulate a point load. This is the same plate that was used to simulate wheel load during the phase two deck testing. A 12 mm thick neoprene pad was placed under the stiffened plate to ensure that load was evenly distributed to the concrete surface below. In addition, a swivel was added at the top of the stiffened plate to allow rotation of the plate when the deck deflected during testing as shown in Figure 4-12.

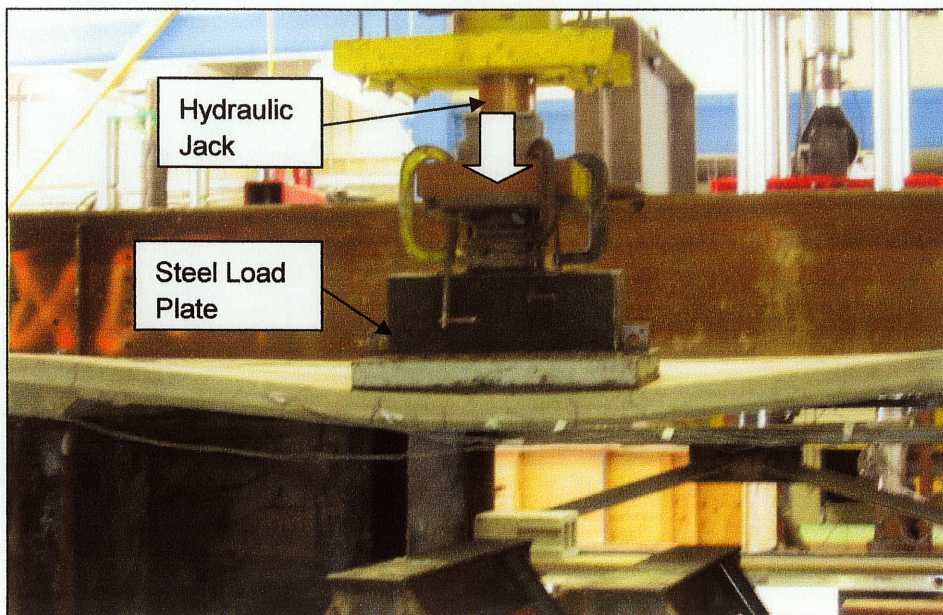


Figure 4-12. Test Set-up for Point Load Test.

4.2.4 *PROCEDURE*

DISTRIBUTED LOAD TESTS

The panels were positioned at the centre of the 9.1 m supporting girders and the edge of the panels rested on neoprene strips supported on the girders top flange. Approximately 75 mm rested on each girder. The panels were loaded up to 25 kN and unloaded. This was repeated five times. The panels were then loaded to 50 kN and held at this load level to

allow time to observe the bottom and top of the panels. This procedure was repeated at load levels of 75 kN, 100 kN and 125 kN.

Once the maximum load had been reached, further load was applied to the panels until they had undergone significant deflection. The load rate was approximately 46 kN/minute for both tests.

POINT LOAD TESTS

The point load tests were carried out at both ends of panel number two. Similar to the distributed load test described above, the panels were loaded to 25 kN and unloaded five times. The panels were then loaded until failure. The load rate was approximately 5.3 kN/minute and 15.6 kN/minute for test numbers one and two respectively.

4.2.5 CONCRETE TESTING

Concrete testing was performed to determine the compressive strength of concrete at the time of testing. A total of five cylinders were cast and tested by Lafarge during the time of testing. The compressive strength of the concrete varied from 40.9 MPa to 44.2 MPa and averaged 41.1 MPa at the time of testing. The standard deviation was 1.7 MPa.

4.3 EXPERIMENTAL PHASE 2 – COMPLETE BRIDGE DECK

4.3.1 DESCRIPTION OF EXPERIMENTAL MODEL

A single deck was constructed during the phase two testing. The deck consisted of three precast panels which were used as stay-in-place forms during the pouring of the upper deck concrete. The ends of the deck were thickened and formed using plywood (Figure 4-13) to support cast-in place concrete. In addition, a transverse 200 mm deep steel channel was placed at the end of the girders above the cross bracing and welded to the underside of the girder's top flange. Nelson studs measuring 22 mm in diameter x 150 mm long were welded to the channel web in accordance with the standard edge stiffening detail provided by the CHBDC (Figure 4-14).



Figure 4-13. Deck Formwork.

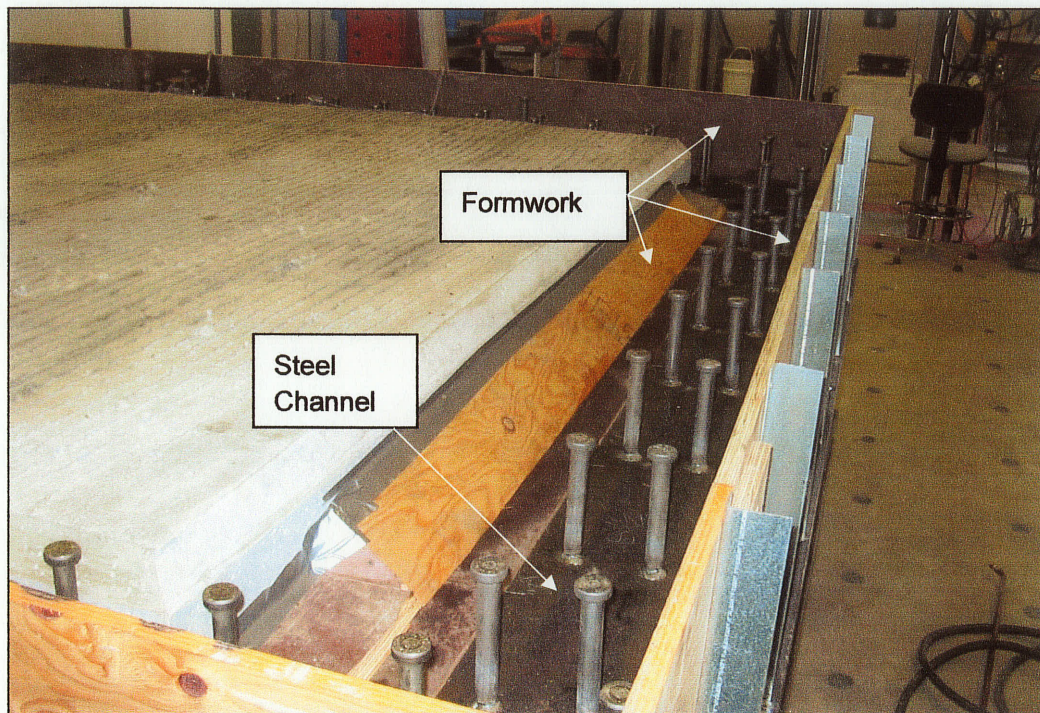


Figure 4-14. Deck end channel and Formwork.

The deck system consisted of three precast panels that were 2,450 mm in width. The outer two panels were 2,400 mm in length and the middle panel was 3,600 mm in length. The cross sectional dimensions were identical to the panels used in the phase one testing program. In addition, the steel straps and Nelson studs were also identical. All panels were reinforced with an orthogonal grid of 10M at approximately 300 mm on centre of GFRP reinforcement. This was the same configuration as panels one and two used in phase one testing. An additional 22 mm diameter x 150 mm long Nelson stud was welded to the portion of the steel strap outside of the precast panel.

Nelson studs measuring 22 mm in diameter x 150 mm long were located in pairs at approximately 170 mm on centre on top of the girder flange (Figure 4-15). These studs were designed in accordance with the CHBDC to ensure composite action between the deck and the girders.

During construction, the panels were set on 25 mm thick neoprene strips that were 25 mm in width. These strips were located at the inside edge of the top flange which provided a 25 mm high x 50 mm wide area under the transverse edge of the panels where concrete grout could flow during the upper deck concrete placement. This ensured that the panels transferred vertical load to the girders by bearing. In addition, 12 mm gaps were placed at 1,200 mm on centre between the neoprene strips. This was done to allow air to escape during the concrete pour.

The top of the precast panels was given a rake finish with a 6 mm amplitude. All edges of the precast panels were chamfered. These surfaces were wetted with water to ensure that they were surface saturated dry when the upper deck concrete was poured.

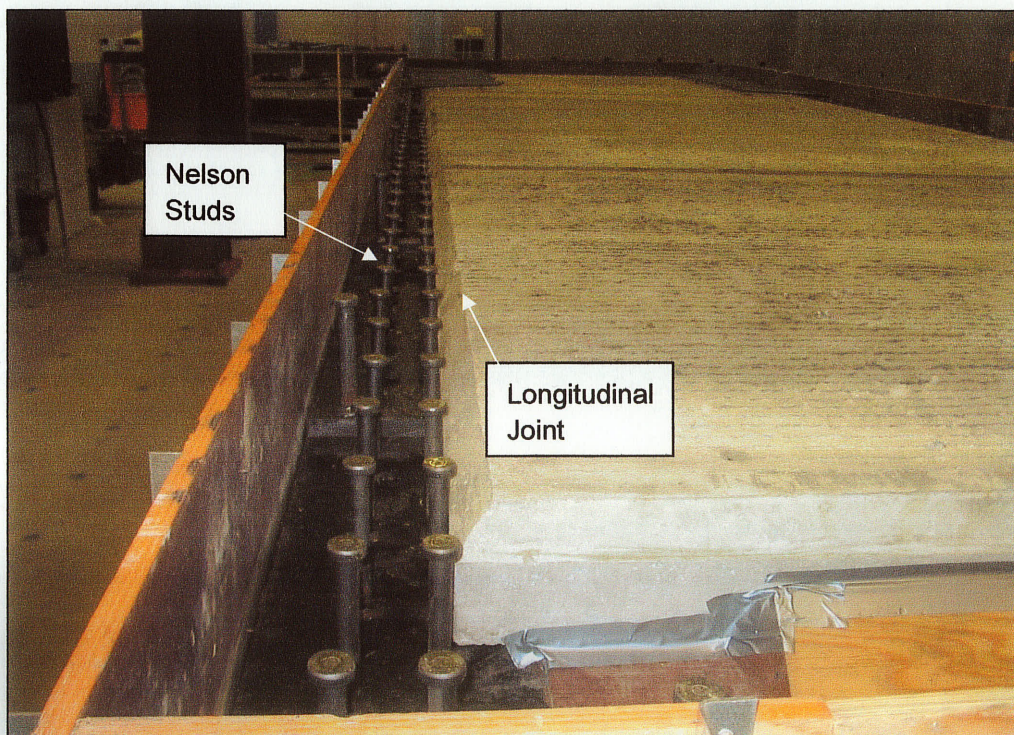


Figure 4-15. Longitudinal Panel Joint and Shear Connectors at Girder.

Stirrups were tied to each pair of Nelson studs located on the top girder flange and spaced at 170 mm. Each stirrup consisted of a 10M and a 15M bar and are shown in Figure 4-16. These stirrups were placed on over one panel section located at the east end of the deck specimen. The purpose of these stirrups was to ensure that the deck did not develop a full depth crack above the longitudinal panel joint which would compromise the composite action and the integrity of the system. This was done since the Nelson studs projecting from the top flange of the girder terminate at the same plane as the top of the precast panel.

Three static point load tests were conducted to simulate axle loading and determine the response of the deck system. Load was applied at the locations shown in Figure 4-17.

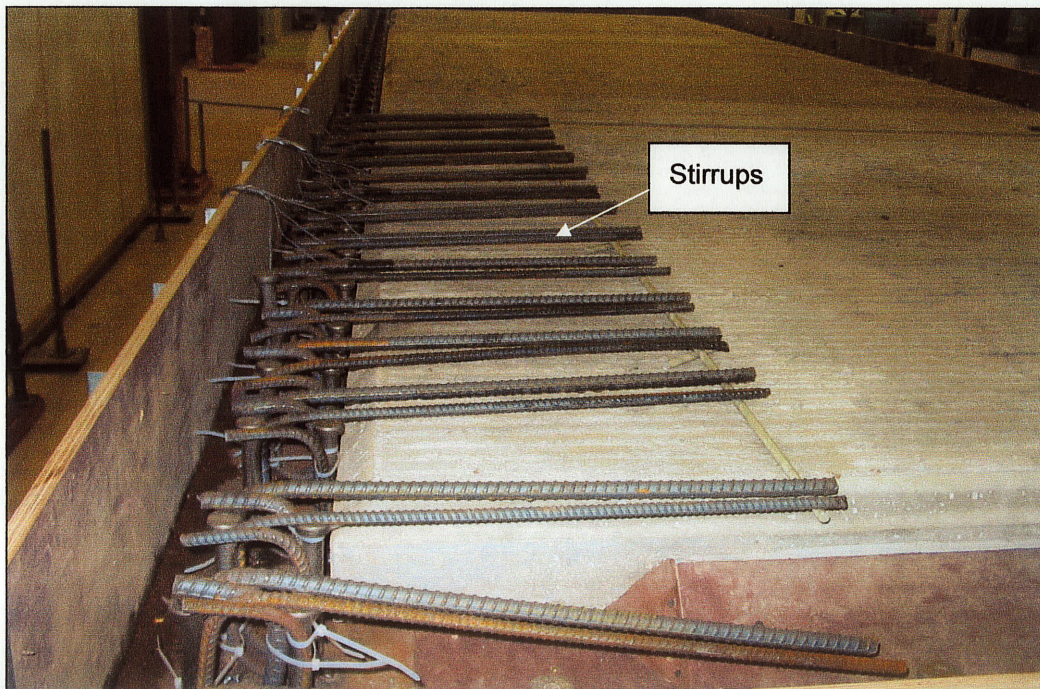


Figure 4-16. Reinforcement Placed at Longitudinal Panel Edge for Panel Test #2.

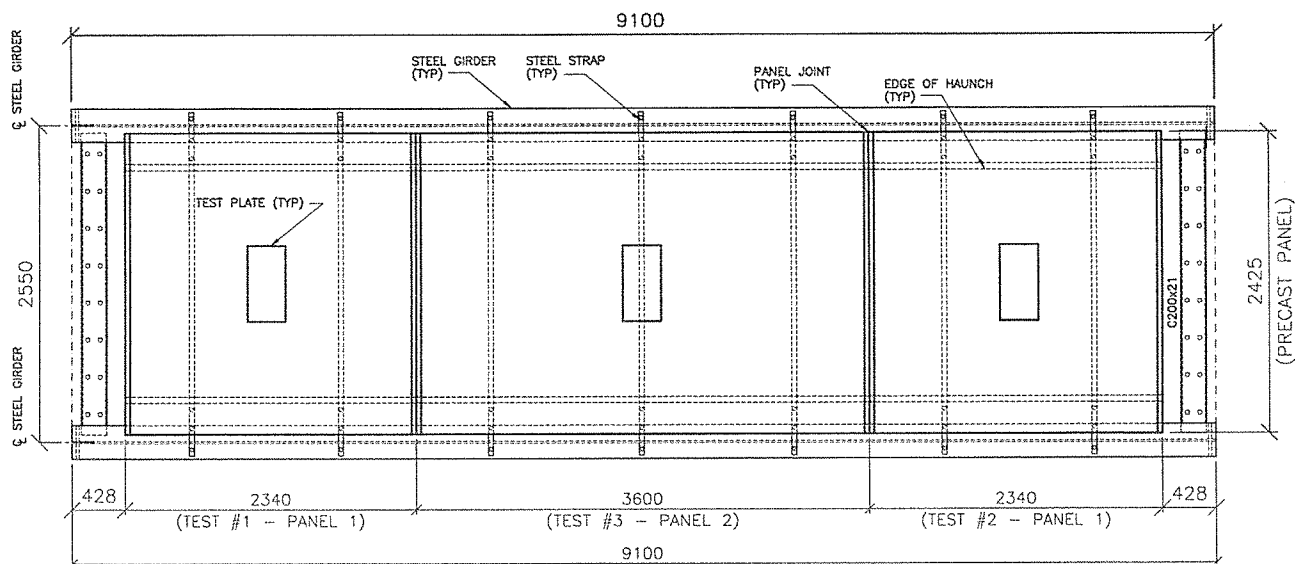


Figure 4-17. Location of Test Load for Deck Test Numbers One to Three.

4.3.2 INSTRUMENTATION OF SPECIMENS

The purpose of the instrumentation of the deck specimens was to monitor the strain in the steel straps and in the GFRP reinforcement. Strain Gauges were also used to measure the strain in the steel stirrups placed over the top flange at the east panel. The purpose of these gauges was to measure the strain in the concrete in the event that a longitudinal crack developed at the longitudinal edge of the concrete panel. Gauges were also installed on the steel girder to ensure that the deck was behaving compositely.

A single strain gauge was placed on the bottom of each of the steel straps at midspan as shown in Figure 4-18. Strain gauges were also placed on the transverse GFRP reinforcement that was cast with the precast panels. These gauges were located at midspan and at span quarter points. Three bars were centred below the wheel load for each test. At the east panel, stirrups were placed over the top flange. A total of six bars were instrumented with two strain gauges as shown in Figure 4-19.

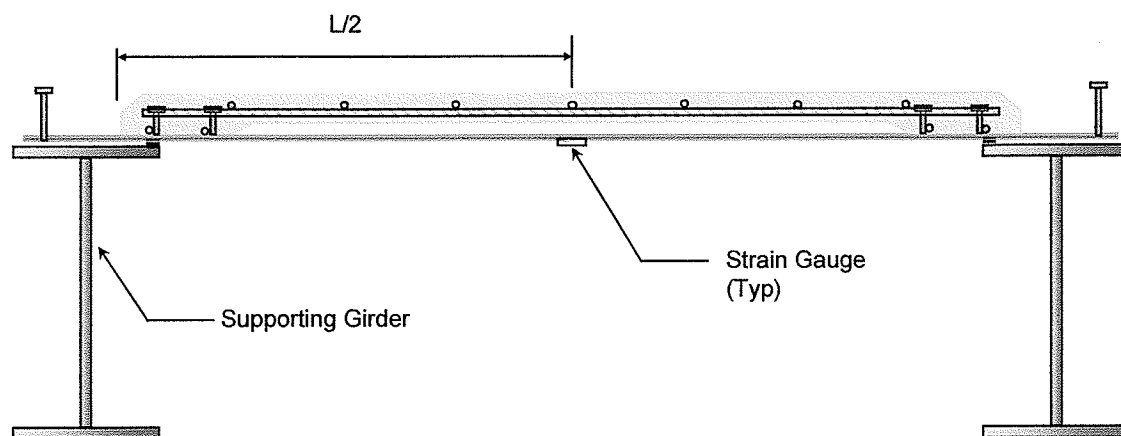


Figure 4-18. Location of Strain Gauge on Steel Strap.

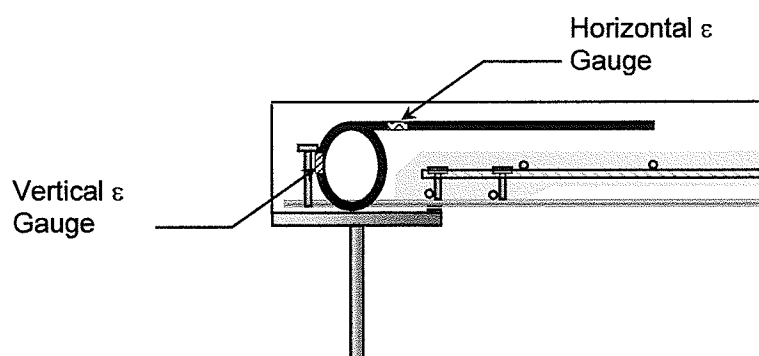


Figure 4-19. Location of Strain Gauges on the Steel Stirrups Located on Panel Number Two.

4.3.3 *TESTING PROCEDURE*

A total of three static tests were performed to determine the failure load and failure mode of the deck. The testing procedure for all tests was identical. The test consisted of ten cycles up to 200 kN and an eleventh cycle to failure. Details were as follows:

Cycle 1 – The deck was loaded in 50 kN increments and the underside of the deck was inspected at load levels of 50 kN, 100 kN, 150 kN and 200 kN. All visible cracks were recorded. Load was held at each level during the visual inspection and then released to 0 kN after inspection of the deck at a load of 200 kN.

Cycles 2, 4, 6, 8, and 10 – The deck was loaded to 200 kN and the underside of the deck was inspected and then the load was released load to 0 kN.

Cycle 3, 5, 7, and 9 – No inspection of the deck occurred during these cycles. The deck was loaded to 200 kN and then released to 0 kN.

Cycle 11 – The deck was loaded to 200 kN and inspected. It was then loaded to 300 kN and inspected and finally loaded until failure.

During all tests, data was collected using Data Acquisition C and a video camera was used to visually record the results. The approximate rate of loading for tests one, two and three were 4.1 kN/second, 2.0 kN/second and 2.4 kN/second respectively.

4.3.4 CONCRETE TESTING

Concrete testing was performed on the concrete cylinders taken during the casting of the precast panels and the cast-in-place concrete deck. The testing of the deck occurred between July 6, 2004 to July 13, 2004 and concrete testing occurred during this time. A total of four cylinders from the deck panels and eight from the deck concrete were broken at the time of testing.

Compressive strength results for the panel concrete varied from 42.3 to 49.9 MPa and averaged 46.6 MPa. Compressive strengths of the cast-in-place deck concrete varied from 40.3 to 47.1 MPa and averaged 43.6 MPa. Tension splitting tests were also conducted on the cast-in-place deck concrete. Results varied from 3.3 to 4.6 MPa and averaged 3.9 MPa. Cylinders were broken the day of the test or within two days of the test. Table 4-2 below provides a summary of the results.

Table 4-2. Summary of Concrete Strengths for the Phase 2 Panels and Deck Concrete.

Concrete Location	Mean Compressive Strength (MPa) and Standard Deviation	Average Tensile Strength (MPa) and Standard Deviation
Panel Concrete	46.6 (2.8)	-
Deck Concrete	43.6 (2.0)	3.9 (0.6)
Total Average	44.6 (2.7)	3.9 (0.6)

5.0 EXPERIMENTAL RESULTS AND DISCUSSION

5.1 EXPERIMENTAL PHASE 1 – DISTRIBUTED LOAD TESTS 1 & 2

Two distributed load tests were performed on panel numbers one and three which are referred to as parallel line load test numbers one and two. During these tests, a maximum load of 142 and 153 kN was observed, respectively at the instant that two longitudinal cracks at the top of the concrete slab developed. The cracks were directly above the concrete haunch near the supports and extended the full length on the specimen on either side.

After the crack had developed, the panels continued to support approximately 120 kN or 86 % of the maximum load under increasing deflection. Shortly after the concrete cracked, the slab continued to deflect until it was resting of the steel straps. Eventually the test was stopped after the deflection exceeded over 65 mm.

5.1.1 *FAILURE MODE AND CRACK PATTERN*

In both tests, failure was initiated by the rupturing or cracking of concrete at the top of the panel at its haunch. The haunched shape and the steel straps caused the edge of the panel to behave as a fixed support. The panels continued to resist this load under increased deflection well after the underside of the concrete panel rested on the steel strap. Note that failure has been defined as the maximum load that the panel supported.

Prior to failure, longitudinal cracks were observed on the underside of the panel. These cracks ran the full length and new cracks were observed as the load was increased. After failure, the cracks on the underside of the panels were spaced at approximately 200 mm on centre throughout the middle two thirds of the panel.

Once the maximum load had been reached, transverse hairline cracks were observed on the top of the panel at each of the steel strap locations. Some additional cracks were at the anchorage of the steel strap and at one location on test number two, localised spalling of the concrete occurred at the anchorage. The concrete spall was approximately 25 mm deep, extending to the top of the reinforcement bar. Details of the cracks are presented in Figures 5-1 and 5-2 below.

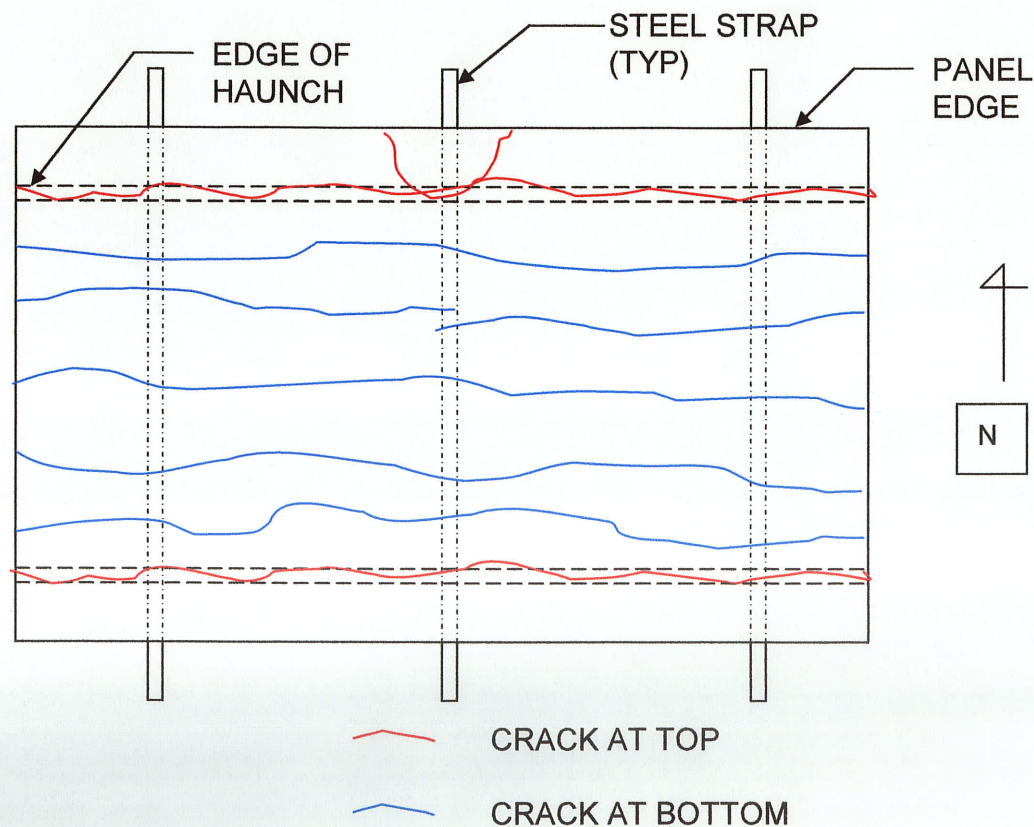


Figure 5-1. Crack Patterns at Failure of Distributed Panel Load Test Number 1.

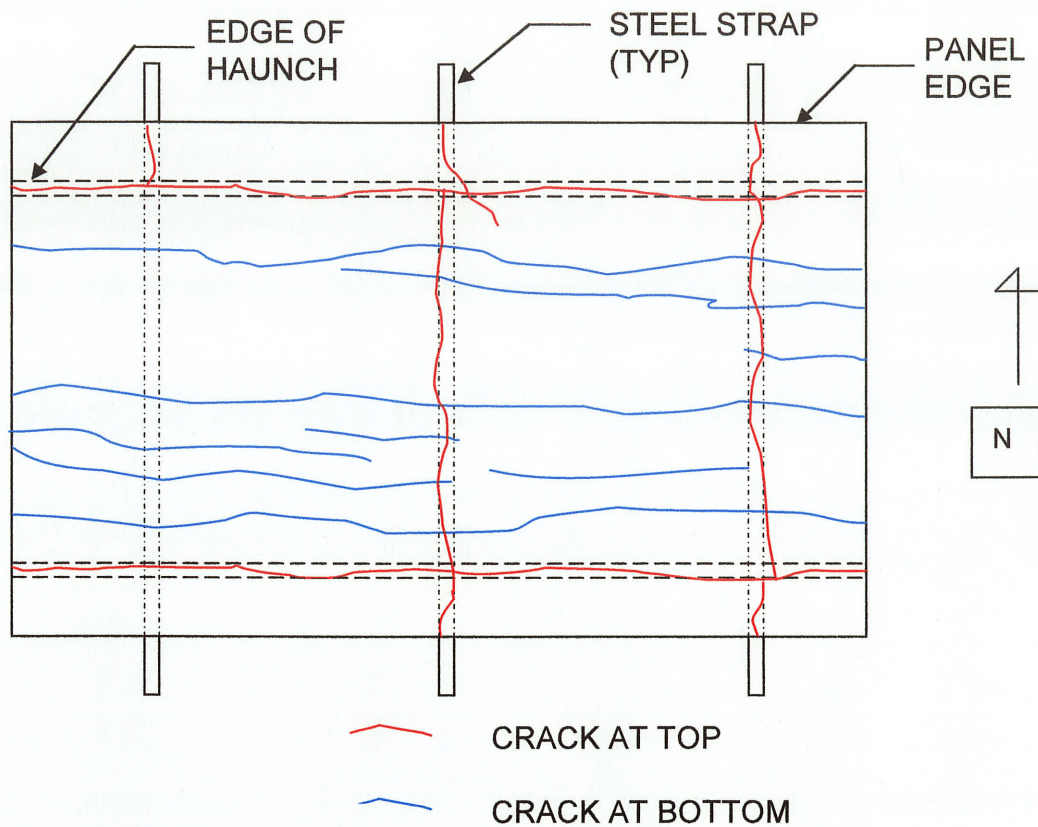


Figure 5-2. Crack Patterns at Failure of Distributed Panel Load Test Number 2.

5.1.2 LOAD STRAIN RESULTS

Strains were monitored in the steel straps and in the transverse GFRP bars located directly above the steel straps. A single strain gauge was mounted on the underside of each steel strap at the mid span. Strain gauges were mounted on the GFRP bars at their mid span and at their quarter points. Bar number one was located over the east steel strap, bar number two was located over the middle steel strap and bar number three was located over the west steel strap.

Figures 5-3 and 5-4 show the load versus strain in the steel straps during both tests. Based on these results it is evident that the panel concrete cracked at approximately 40 kN. This is the point at which the slope on the load strain curve changes and compares well with observations during the test. A longitudinal crack was observed at 50 kN at the midspan of the panel.

The CHBDC and AASHTO Guide Design Specifications for Bridge Temporary Works service and factored load have been shown on the graphs. In all cases, the panel capacity is well above the calculated loading. The strains in the steel straps under factored CHBDC and AASHTO load correspond to vary between 94 and 406 μ strain, respectively (Table 5-1). In all cases and at failure the strap strains are well below the yield strain of the steel strap.

Table 5-1. Strain in Steel Straps at CHBDC and AASHTO Loading for Panel Distributed Load Tests.

Load Type and Value (kN)	Steel Strap Strain (micro strain)						
	Test #1			Test #2			Mean
	East End	Middle	West End	East End	Middle	West End	
CHBDC Service Load P = 30.1 kN	114	117	110	79	66	69	93
AASHTO Service Load P = 33.1 kN	127	128	121	91	76	80	104
CHBDC Factored Load P = 38.1 kN	154	154	143	107	89	94	124
AASHTO Factored Load P = 82.8 kN	406	402	370	363	337	321	367

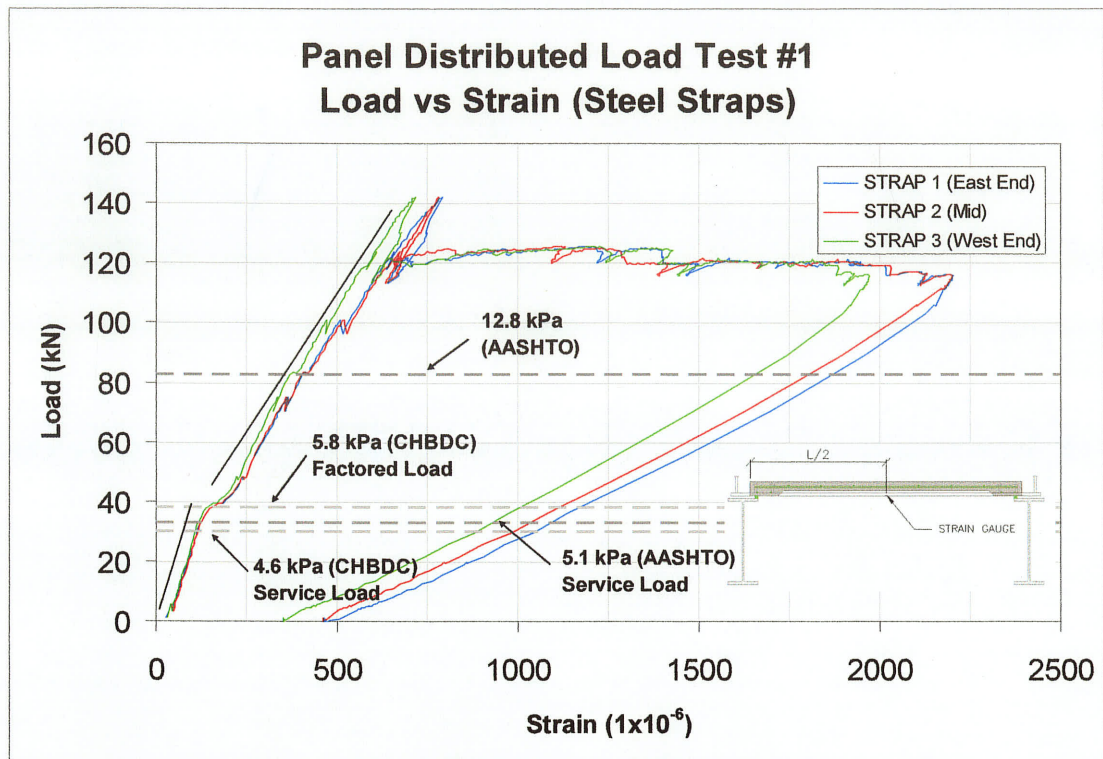


Figure 5-3. Panel Distributed Load Test #1 – Load Versus Strain in Steel Straps.

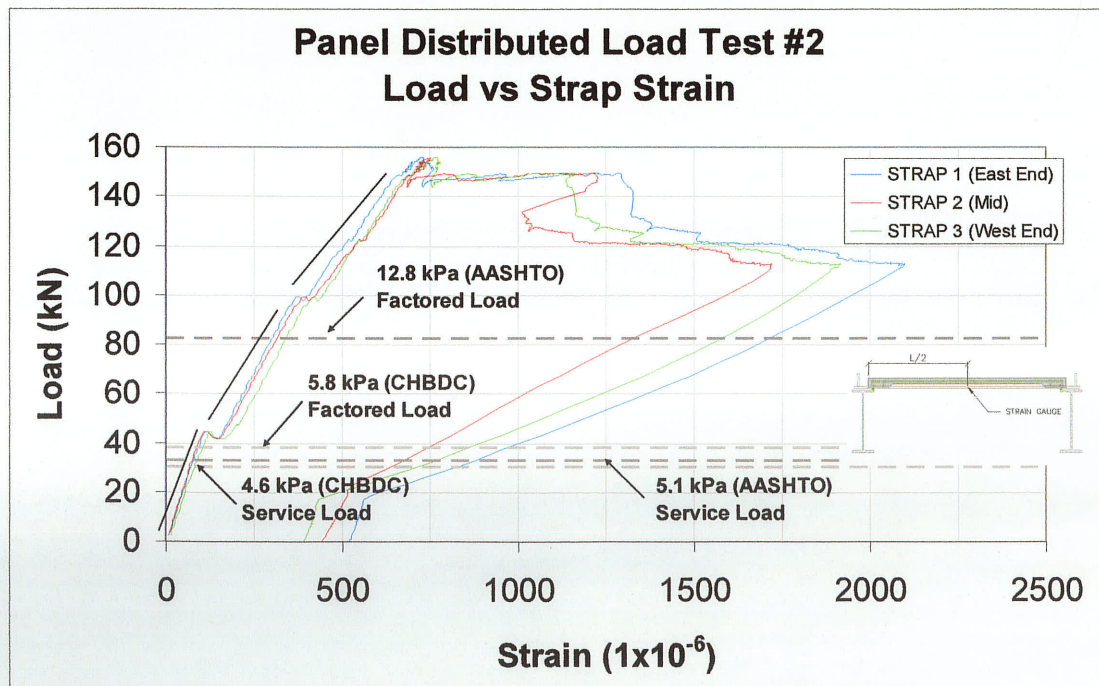


Figure 5-4. Panel Distributed Load Test #2 – Load Versus Strain in Steel Straps.

Strain was monitored in the transverse GFRP rebars in addition to the steel straps. Figures 5-5 to 5-10 illustrate the load versus strain behaviour in the GFRP bars. The load versus strain behaviour of strap #1 from test #1 has been shown on these graphs as a reference point. Results showed that the GFRP resisted very little strain compared to the strain resisted by the steel straps. Therefore, it is evident that the steel straps carried the load.

The maximum strain in the GFRP at the factored AASHTO design criteria was 200 μ Strain. Below this load, all GFRP bars resisted a negligible amount of strain since these bars are very close to the neutral axis of the specimen. After failure and continual deflection of the panels, the strain in some of the GFRP bars began to increase. This occurred after the concrete had cracked.

Although the steel straps are not fully bonded to the concrete panels along their length, it is evident that they are resisting load through bending behaviour with the concrete panels. In addition, the strains in the bars are minimal indicating that they can be fully utilised to control cracking.

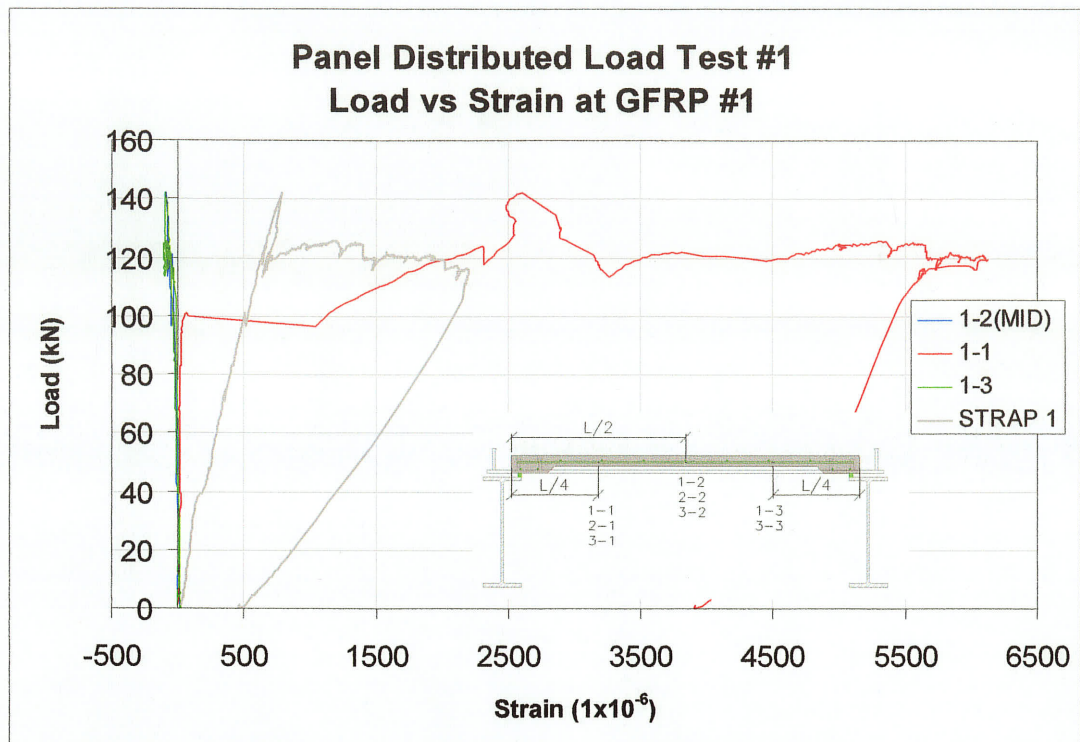


Figure 5-5. Panel Distributed Load Test #1 – Load Vs. Strain at GFRP Bar #1.

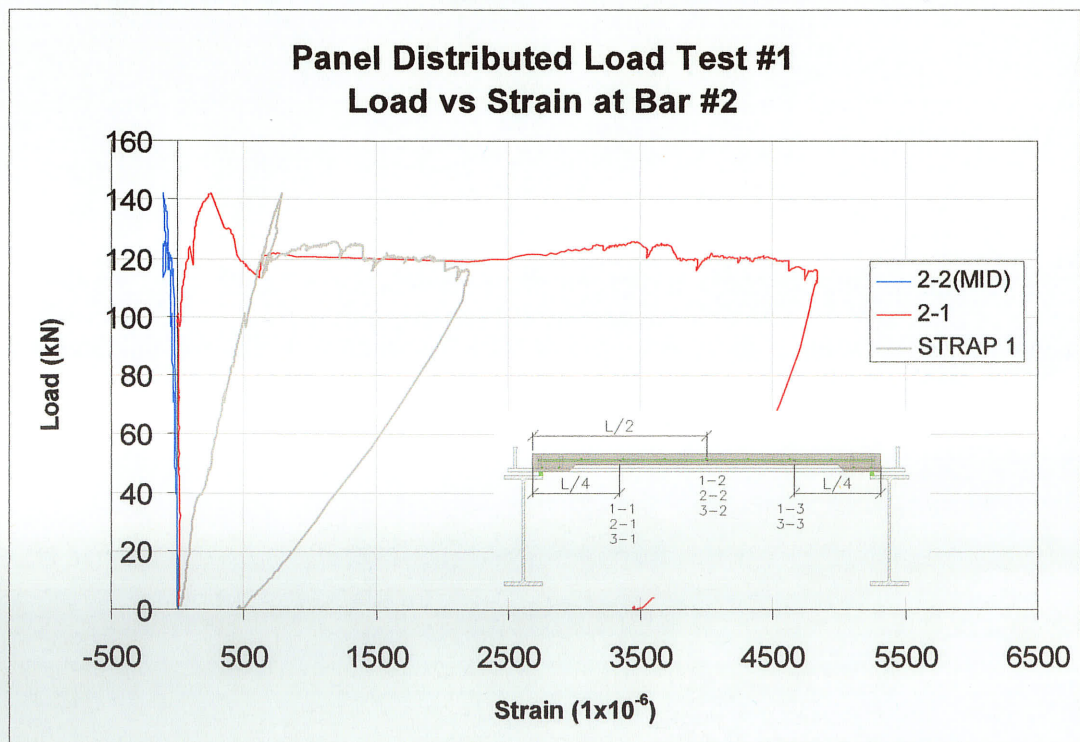


Figure 5-6. Panel Distributed Load Test #1 – Load Vs. Strain at GFRP Bar #2.

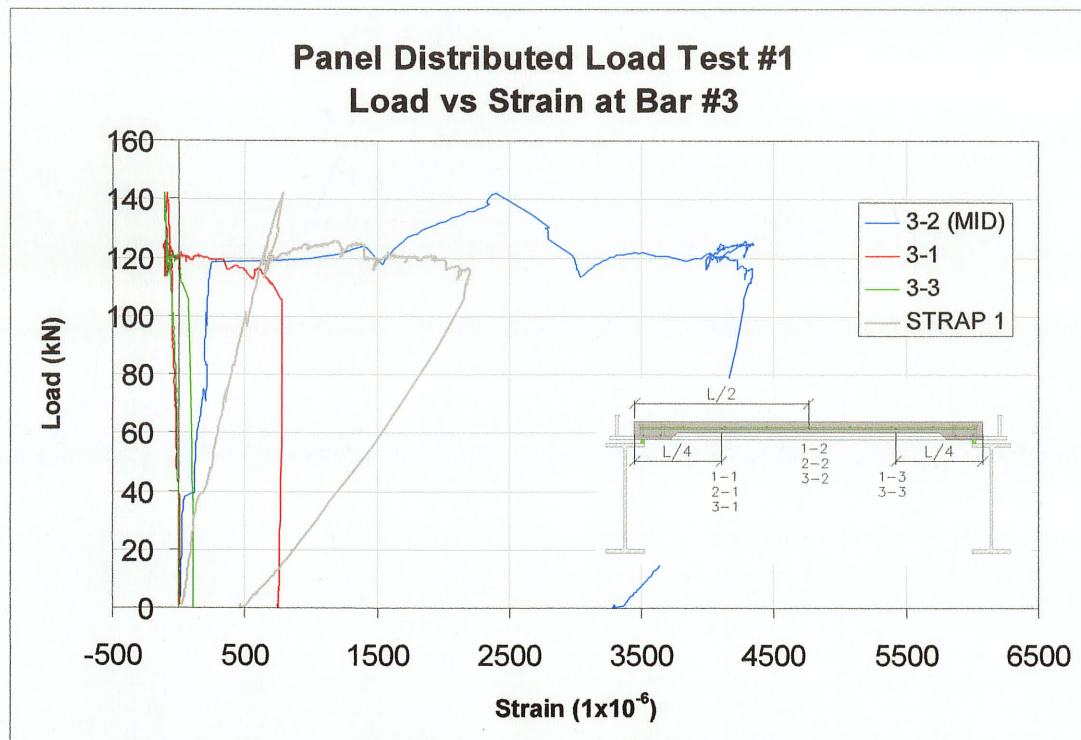


Figure 5-7. Panel Distributed Load Test #1 – Load Vs. Strain at GFRP Bar #3.

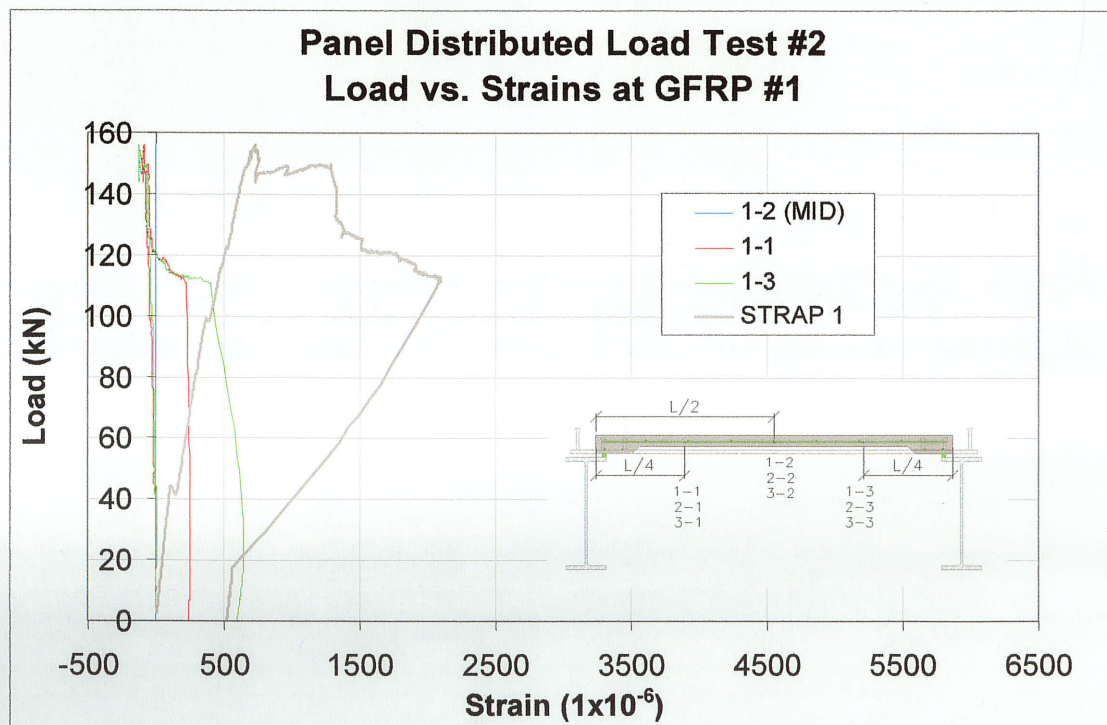


Figure 5-8. Panel Distributed Load Test #2 – Load Vs. Strain at GFRP Bar #1.

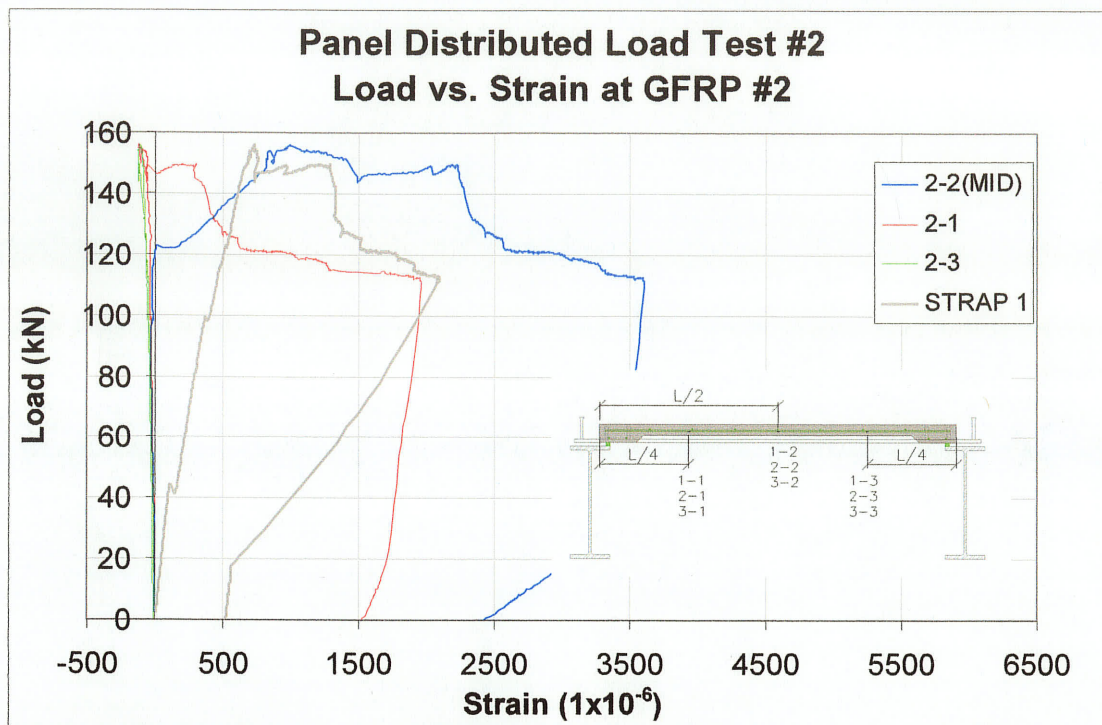


Figure 5-9. Panel Distributed Load Test #2 – Load Vs. Strain at GFRP Bar #2.

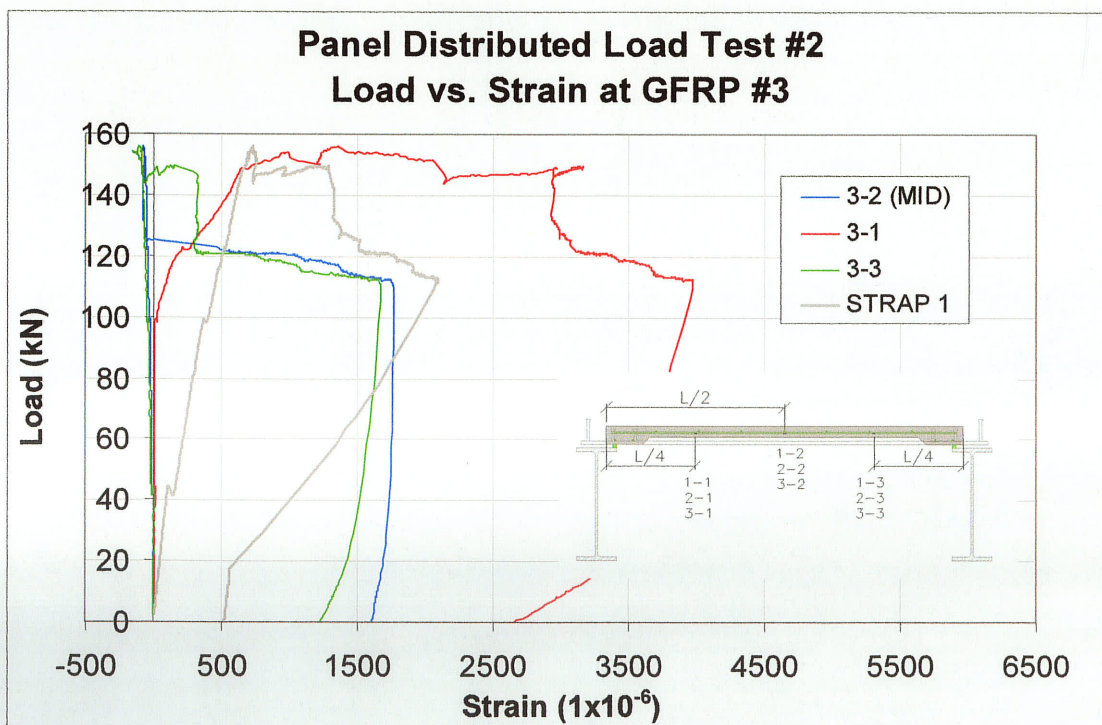


Figure 5-10. Panel Distributed Load Test #2 – Load Vs. Strain at GFRP Bar #3.

5.1.3 LOAD DEFLECTION RESULTS

Three LVDTs were placed on each panel to measure the deflection of the panels during the testing. The LVDTs were located directly over the steel straps at the mid span. Table 5-2 and Figures 5-11 to 5-12 show the results for tests one and two. The first change in slope of the load deflection curve was at approximately 40 kN similar to the curve showing the load versus steel strap strain. The slope of the load deflection curve is nearly linear after 40 kN until failure.

The average deflection of the panels at the CHBDC and AASHTO Service loads was 1.5 mm and 1.6 mm respectively which corresponds to L/1617 and L/1516. The actual mass of the cast-in-place concrete corresponds to a load equal to 2.35 kPa or a 15 kN load when comparing evenly distributed loads to line loads used during the actual testing. At this load the deflection is less than 1.0 mm

The deflection of the panel at the AASHTO factored load was 6.5 mm and 4.7 mm for test numbers one and two. These results are approximately equal to the typical cast-in-place construction tolerances that are +/- 6 mm and are considered negligible.

Table 5-2. Panel Distributed Load Tests Deflection at CHBDC and AASHTO Service and Factored Loads.

Load	Deflection (mm)		
	Test #1	Test #2	Average
CHBDC Service Load P = 30.1 kN	1.9	1.1	1.5 (L/1617)
AASHTO Service Load P = 33.1 kN	2.0	1.2	1.6 (L/1516)
CHBDC Factored Load P = 38.1 kN	2.4	1.4	1.9 (L/1276)
AASHTO Factored Load P = 82.8 kN	6.5	4.7	5.6 (L/433)

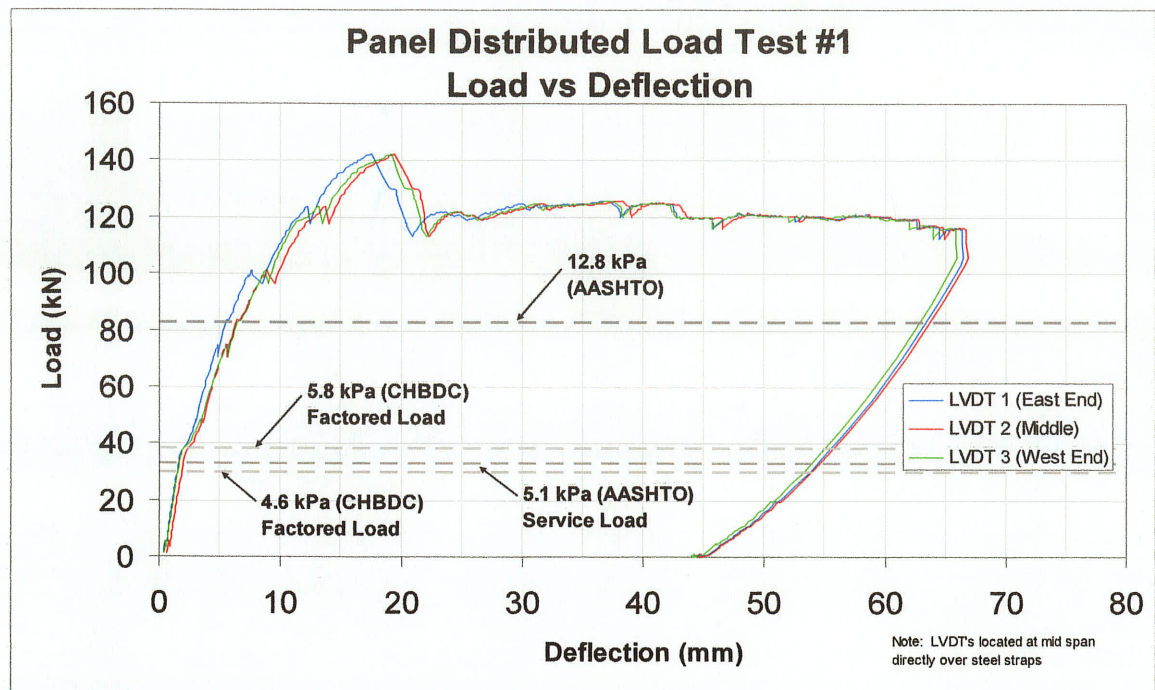


Figure 5-11. Panel Distributed Load Test #1 – Load Vs. Deflection.

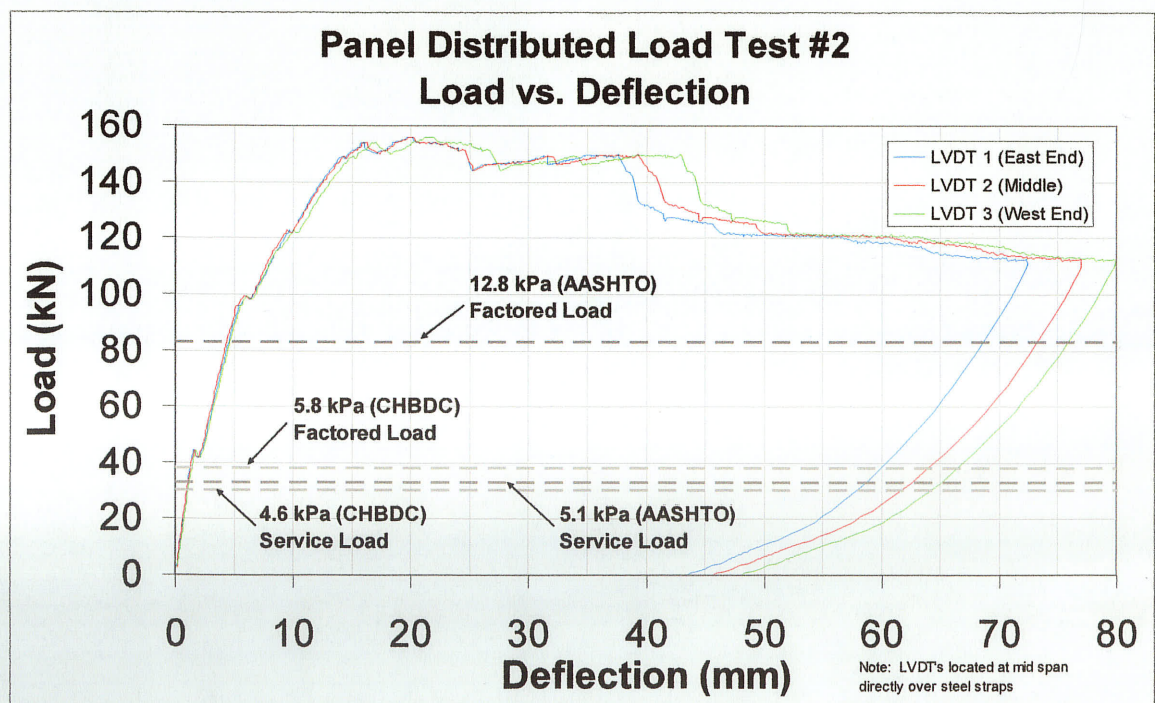


Figure 5-12. Panel Distributed Load Test #2 – Load Vs. Deflection.

5.2 EXPERIMENTAL PHASE 1 – POINT LOAD TESTS 1 & 2

Panel two was tested under a static point load test at either end of the specimen. A stiffened steel plate measuring 305 x 610 mm was positioned at panel edge at the centreline. The panels failed at 29 and 32 kN under the point load and the failure pattern was a semi-circular crack at the top surface of the concrete.

5.2.1 *FAILURE MODE AND CRACK PATTERN*

The two deck specimens failed in more of a shear failure than a bending failure. A semi-circular crack developed on the top of the concrete as shown in Figure 5-13 and 5-14. There was no distinct failure point other than the crack width continued to increase under increasing load.

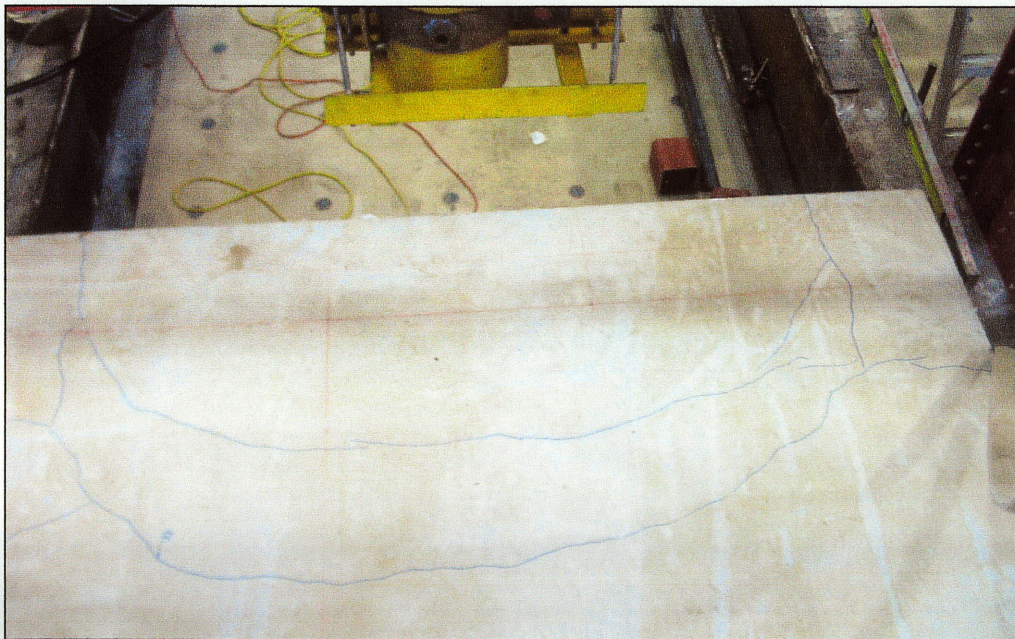


Figure 5-13. Crack Patterns on Top of Concrete Panel at Failure - Panel Point Load Test #1.

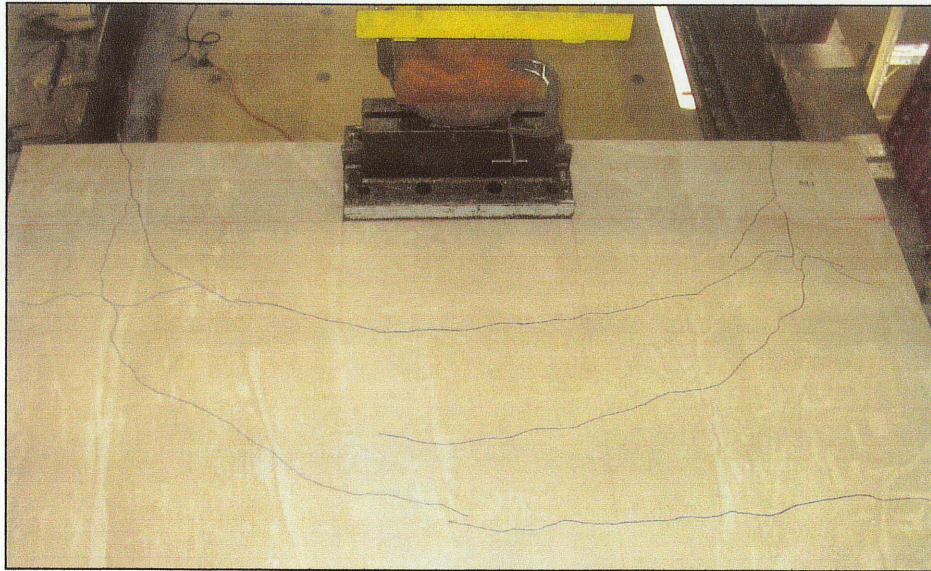
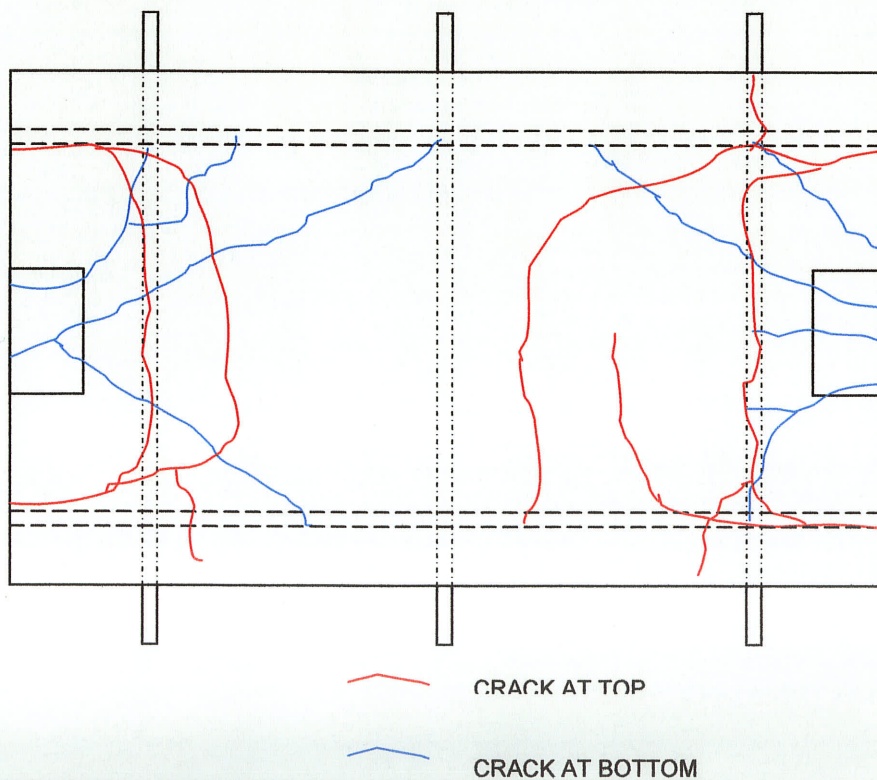


Figure 5-14. Crack Patterns on Top of Concrete Panel at Failure - Panel Point Load Test #2.



POINT LOAD TEST #1

POINT LOAD TEST #2

Figure 5-15. Crack Pattern at Failure for Panel Point Load Test Numbers 1 and 2.

5.2.2 *LOAD VERSUS STRAIN RESULTS*

Strain was measured in the steel straps, in a transverse GFRP bar located mid span between the transverse free edge of the panel and the end steel strap and in five longitudinal GFRP bars. Figures 5-16 to 5-17 show the results of point load tests one and two.

A slight change in slope of the load versus strain in the steel straps curve was observed in both tests at approximately 14 kN. Crack patterns at failure indicate that the point load was resisted in two way flexure. Negligible strain was measured in the transverse and longitudinal bars until after failure or the maximum load had been reached.

Strain measurements in the steel strap indicate that it resisted the majority of the transverse flexural load even though it was not bonded to the concrete slab along its full length. Tables 5-3 and 5-4 below summarise the deflection and strains in the steel straps and GFRP reinforcement.

Table 5-3. Panel Deflection, Strain in Steel Straps and Strain in Transverse GFRP Bar at Maximum Load.

Panel Test #	Maximum Load (kN)	Deflection at Maximum Load (mm)		Strain in Steel Straps at Maximum Load (μ strain)		Strain in Transverse GFRP Bar over End Steel Strap at Maximum Load (μ strain)		
		Panel Edge	Inner LVDT	Edge Strap	Middle Strap	End	Middle	End
1	29.3	18.3	8.8	333	125	-26	-8	-41
2	32.3	8.1	5.2	256	126	-61	1091	-64

Table 5-4. Strain in Longitudinal GFRP Bars at Maximum Load.

Panel Test #	Maximum Load (kN)	Strain in Longitudinal GFRP Bars Measured above Edge Steel Strap (μ strain)				
		L1 (Edge)	L2 (Middle Edge)	L3 (Middle)	L4 (Middle Edge)	L5 (Edge)
1	29.3	1195	109	72	318	49
2	32.3	24	38	67	48	42

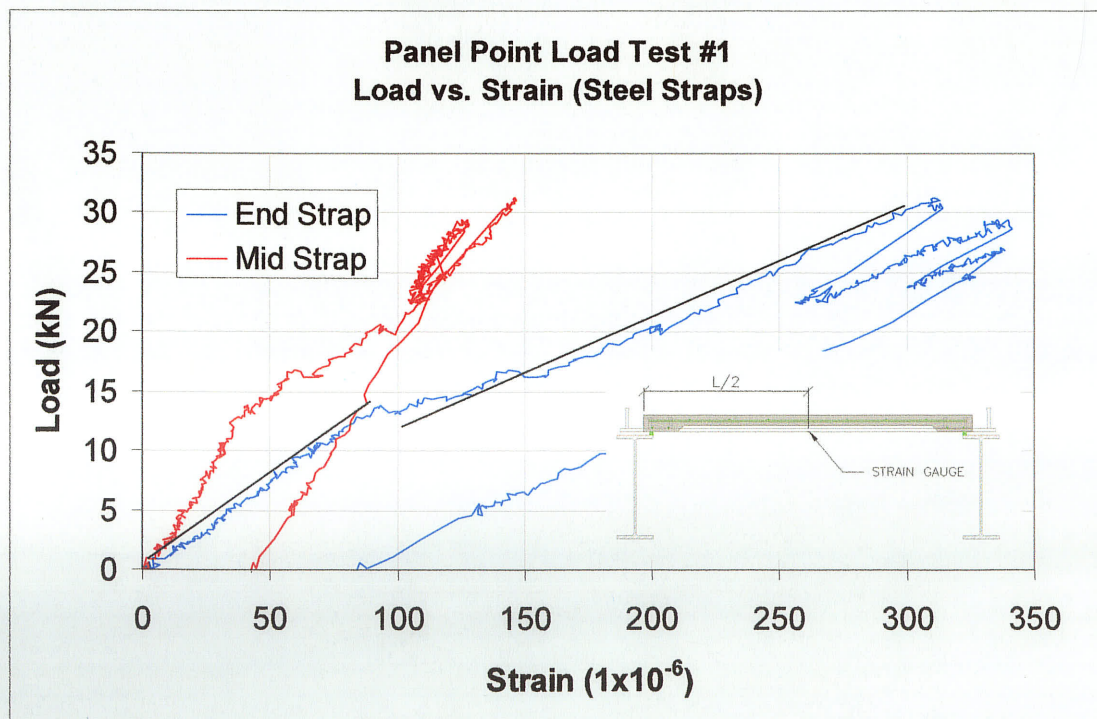


Figure 5-16. Panel Point Load Test #1 – Load Vs. Strain in the Steel Straps.

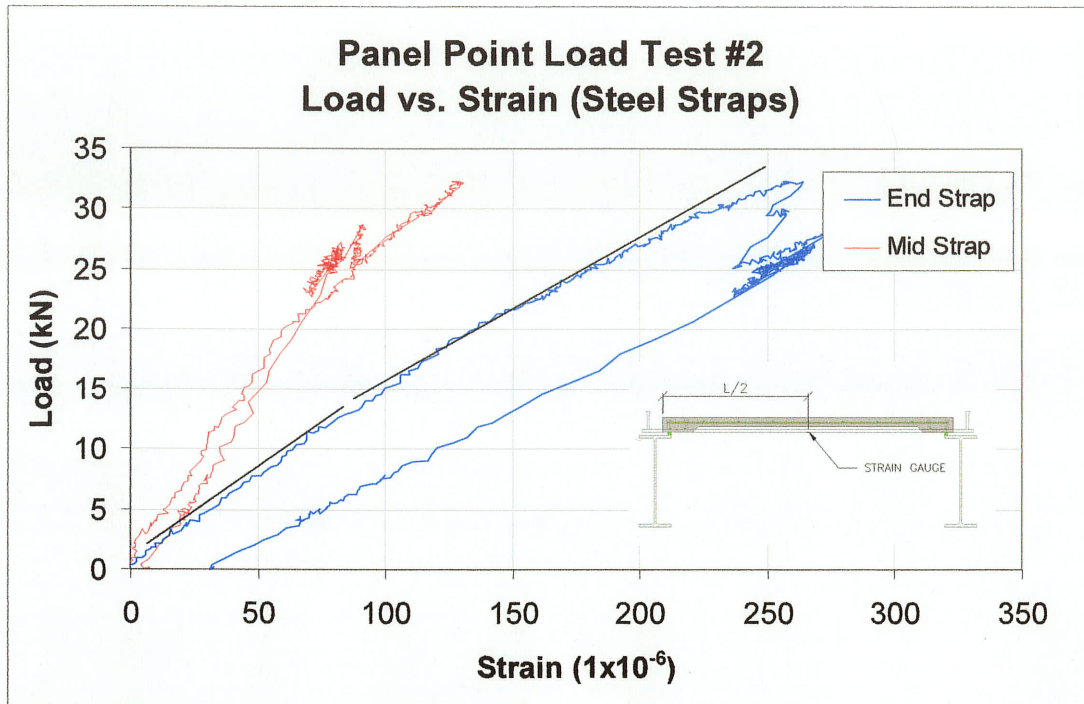


Figure 5-17. Panel Point Load Test #2 – Load Vs. Strain in the Steel Straps.

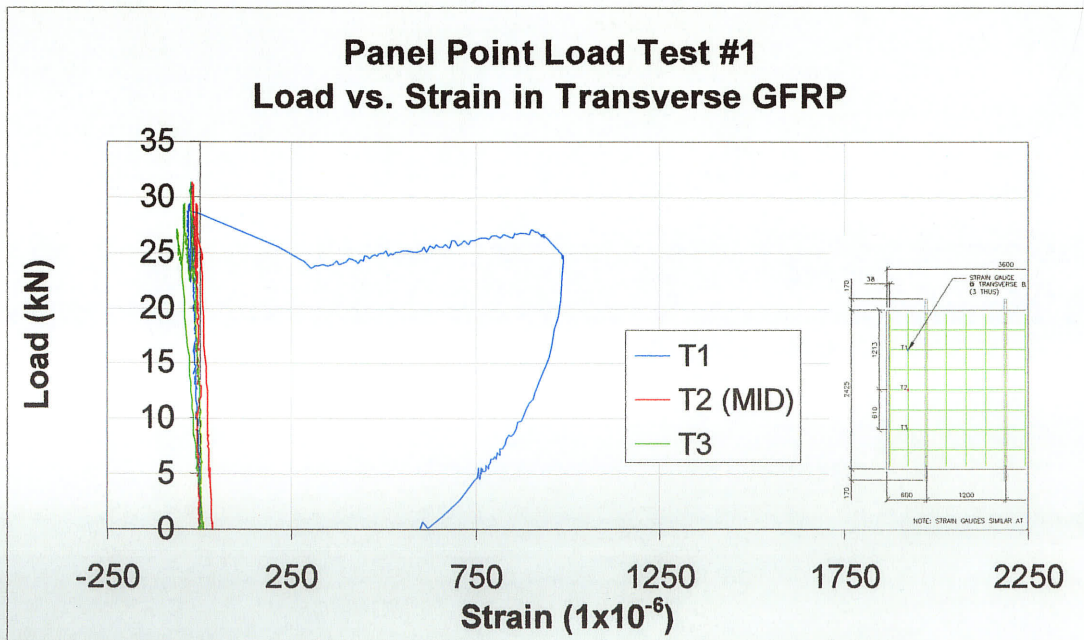


Figure 5-18. Panel Point Load Test #1 – Load Vs. Strain in the Transverse Bars.

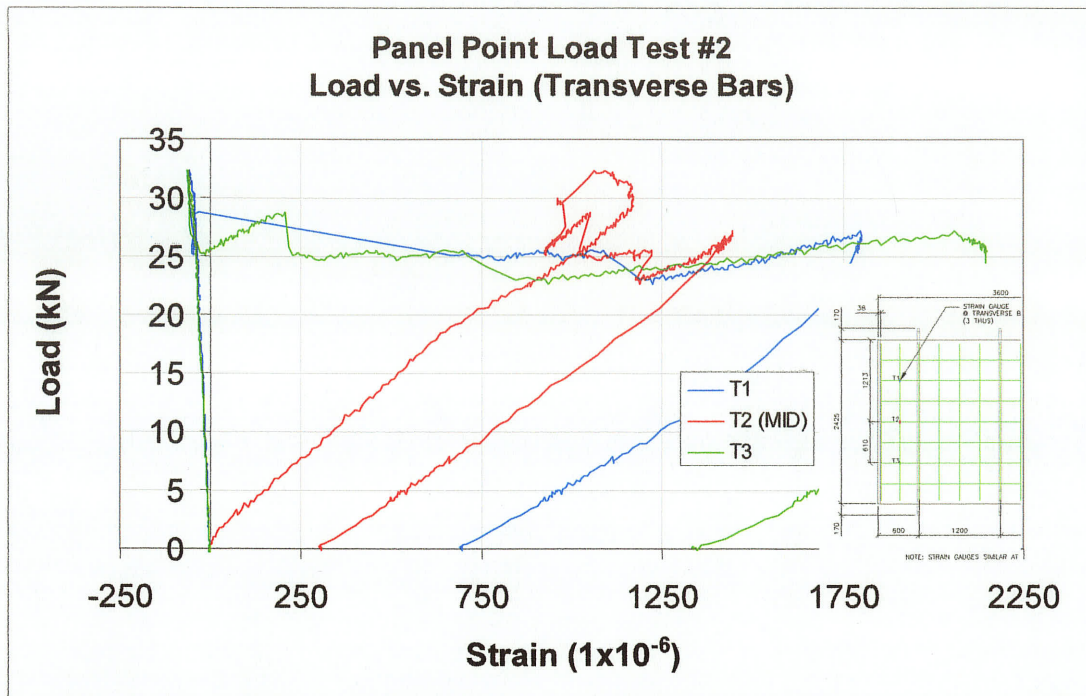


Figure 5-19. Panel Point Load Test #2 – Load Vs. Strain in the Transverse Bars.

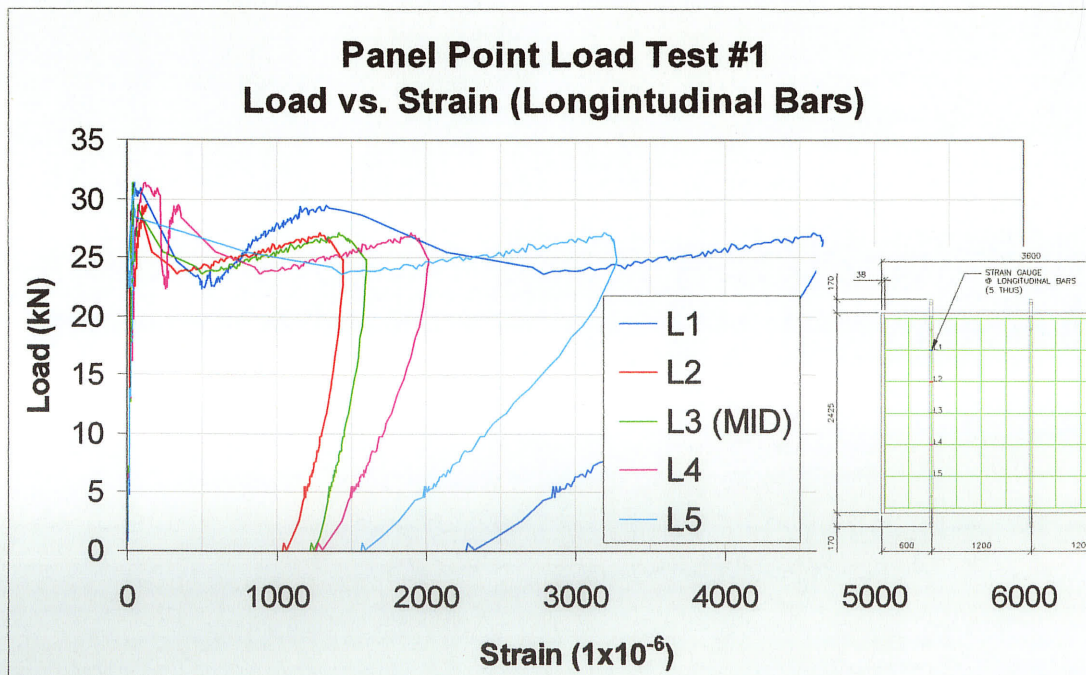


Figure 5-20. Panel Point Load Test #1 – Load Vs. Strain in the Longitudinal Bars.

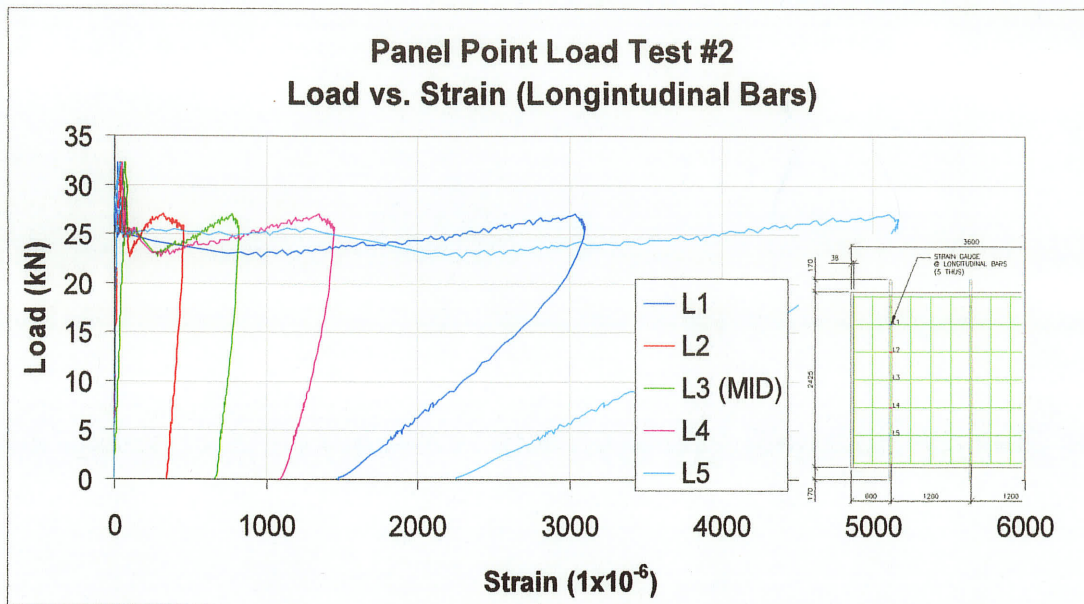


Figure 5-21. Panel Point Load Test #2 – Load Vs. Strain in the Longitudinal Bars.

5.2.3 LOAD DEFLECTION RESULTS

Panel deflection was measured at the midspan of the concrete panels on either side of the load plate. One LVDT was at the free edge of the panel and the other was located approximately 400 mm inside of the free edge at the panel centreline. Results are shown in Figures 5-22 and 5-23. Based on these test results, it appears that the first crack in the panel occurred at a load of approximately 14 kN. This is indicated by the change in slope of the load deflection curve. This result was not observed during test #2 where the first change in slope of the load deflection curve occurred at approximately 22 kN.

During both tests, deflection was recorded at both the edge of the panel and the LVDT located adjacent the inside edge of the load plate. The difference between these measurements was approximately 2 to 1. This further confirms that the panel was resisting the load in two way action.

Once the maximum load was reached the load dropped to 23 kN. Loading continued and the panels were able to resist about 28 kN, just below its original maximum load. The load dropped off at this point to about 23 kN and then under increasing deflection the panel continued to resist additional load until it reached 27 kN. The test was terminated at this point since the edge of the panel had deflected more than 25 mm (Figures 5-22 to 5-23).

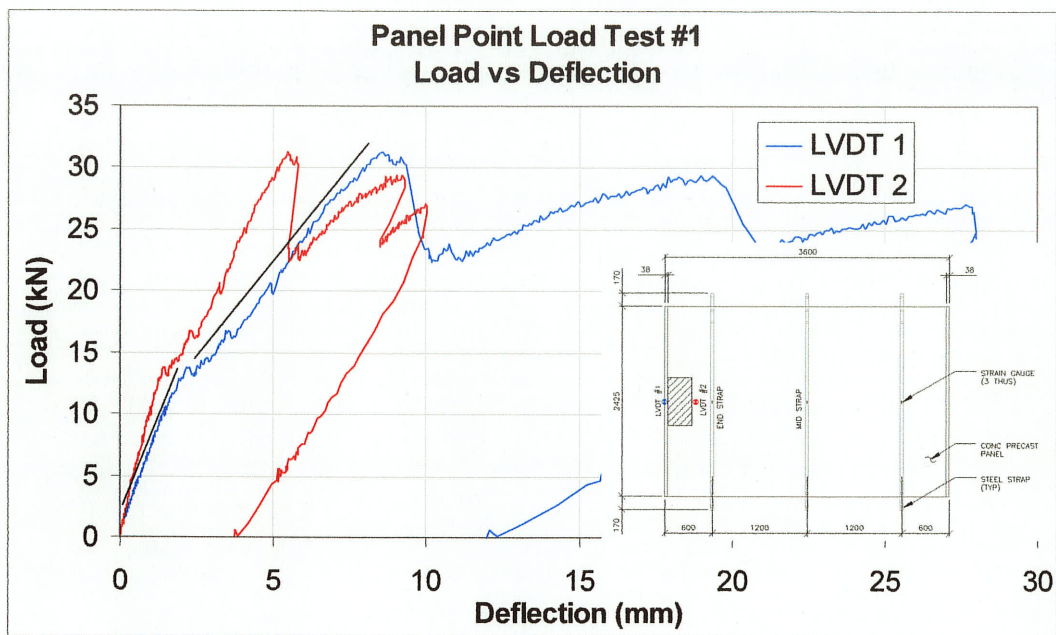


Figure 5-22. Panel Point Load #1 – Load Vs. Deflection.

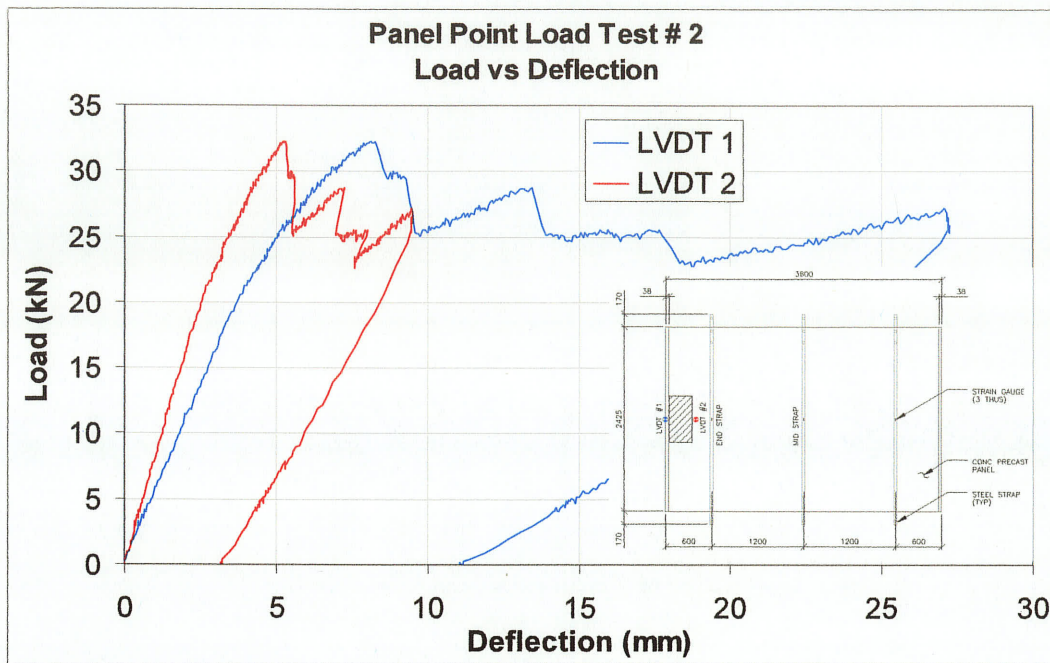


Figure 5-23. Panel Point Load #2 – Load Vs. Deflection.

5.3 EXPERIMENTAL PHASE 2 –BRIDGE DECK

Three identical tests were conducted at different locations on the deck specimen. Each of the static wheel load tests were carried out until failure and all specimens failed by punching shear. The failure loads of the specimens ranged from 573 kN to 632 kN.

5.3.1 FAILURE MODE AND CRACK PATTERN

A typical punching shear failure occurred during all three tests and similar crack patterns developed as shown in Figures 5-24 to 5-25. The first visible cracks observed were hairline longitudinal cracks that were near the midspan. A hairline transverse crack directly under the test plate was also observed. These cracks appeared at approximately 200 kN. As the load increased additional longitudinal cracks developed as well as radial cracks. This is occurred near the failure load and is consistent with punching shear failures.

Sudden failure occurred by punching shear in all specimens. The failure was punctuated by a loud bang as the concrete failed in shear. The failure area extended from the load plate at the top of the deck to the inside edge of the concrete haunch at the soffit. In all cases the concrete failed through the 175 mm thick portion of the deck not the thicker 250 mm portion over the girders.

The deck was diamond cut into three sections when it was removed from the laboratory which allowed inspection of the horizontal joint between the precast panel concrete and cast-in-place concrete. There was no visible evidence that the precast panel concrete had separated from the cast-in-place deck concrete.

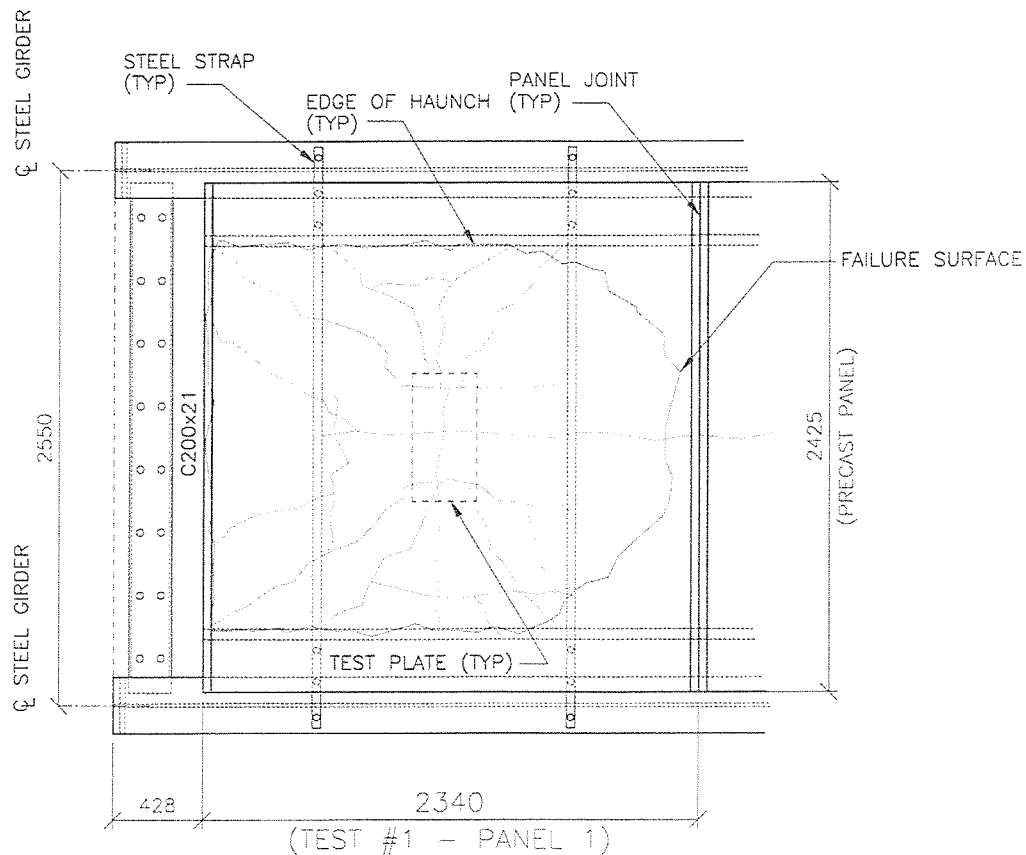


Figure 5-24. Crack Pattern at Underside of Deck at Failure – Deck Test #1.

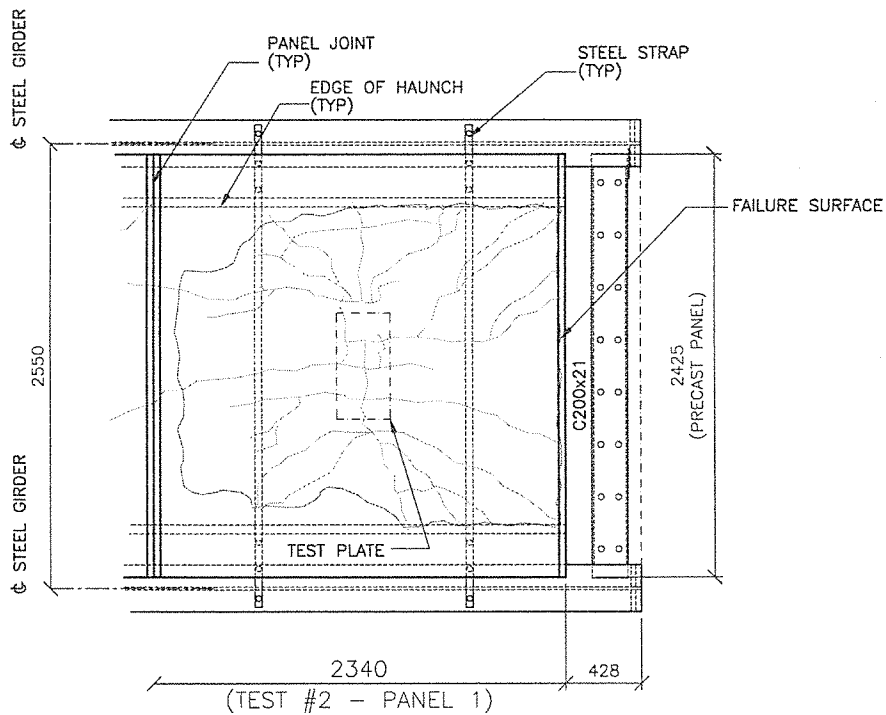


Figure 5-25. Crack Pattern at Underside of Deck at Failure – Deck Test #2.

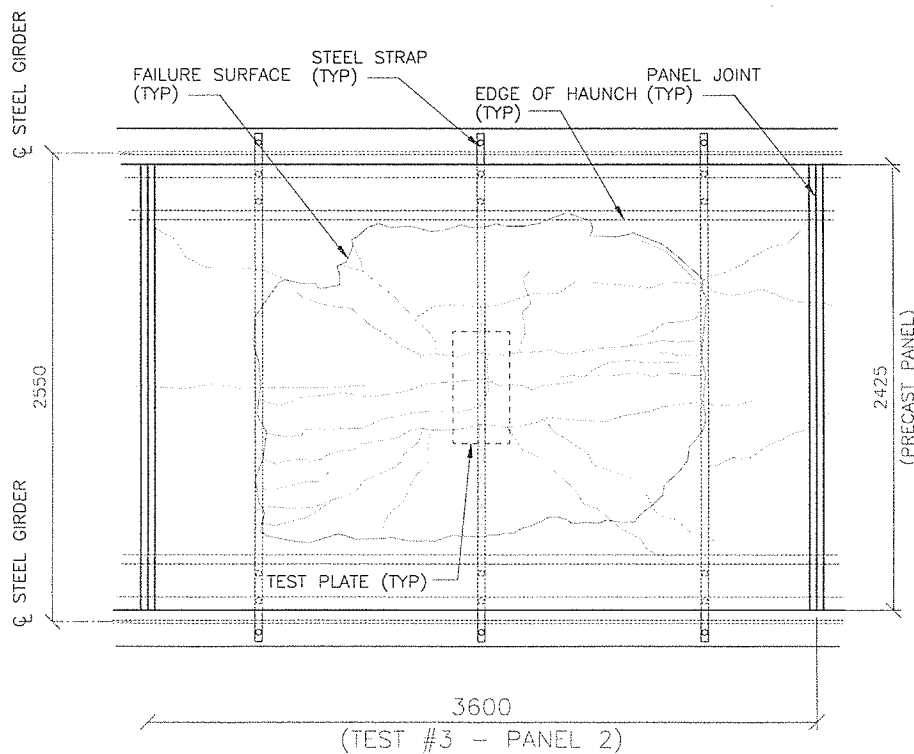


Figure 5-26. Crack Pattern at Underside of Deck at Failure – Deck Test #3.

5.3.2 LOAD VERSUS STRAIN IN STEEL STRAPS RESULTS

Load versus strain results for tests one to three are summarised in Table 5-5 and shown in Figures 5-26 to 5-28. The average failure load was 606 kN and the average failure strain of the straps adjacent the wheel load ranged from 1346 micro strain to 1476 micro strain. The load versus strap strain curves were very similar in all three tests.

Table 5-5 Deck Tests Failure Load Versus Failure Strain.

Deck Test Number	Ultimate Load (kN)	Failure Strain (μ Strain)	Average Failure Strain (μ Strain)
Test #1	642	Strap #1 – 1436 Strap #2 – 1414	1425
Test #2	573	Strap #5 – 974 Strap #6 – 1407 Strap #7 – 1545	1476 (Note: Straps #6 and 7)
Test #3	603	Strap #3 – 1435 Strap #4 – 1320 Strap #5 – 1283 Strap #6 – 1099	1346 (Note: Straps #3 to 5)

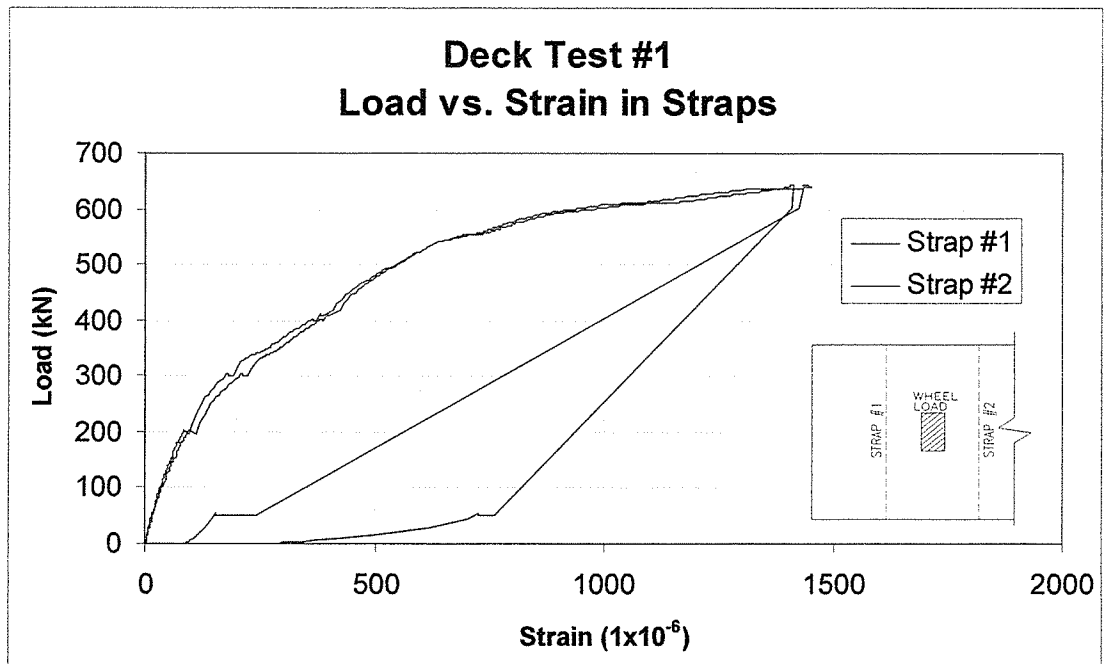


Figure 5-27. Deck Test #1 – Load vs. Strain in Straps.

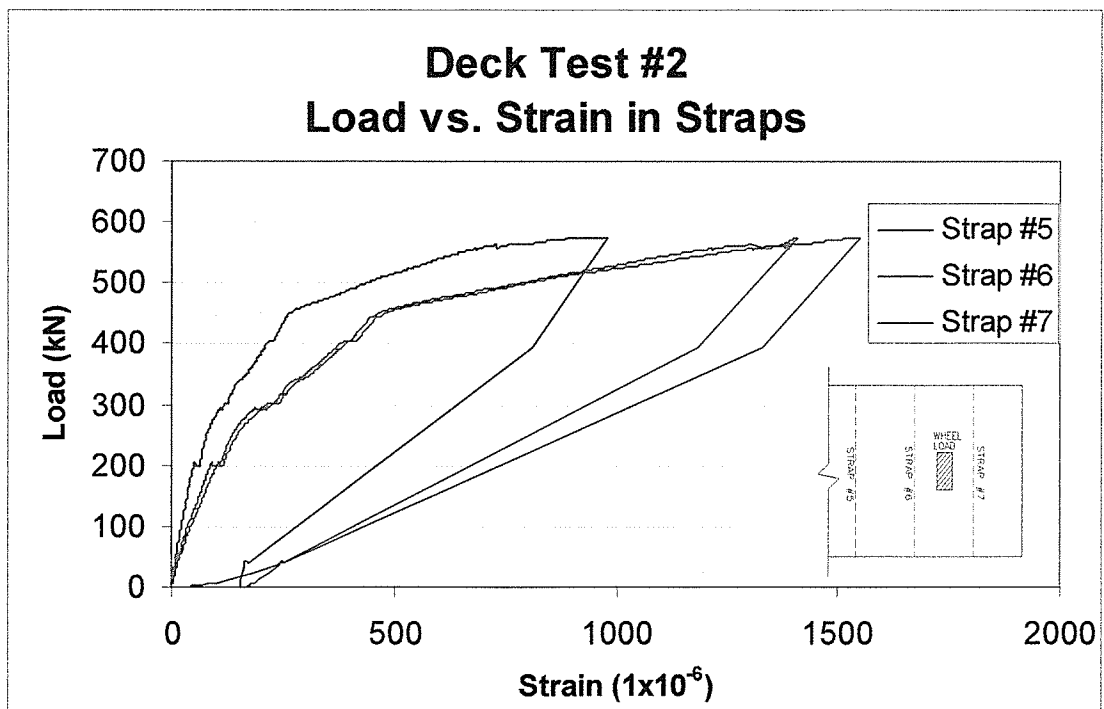


Figure 5-28. Deck Test #2 – Load vs. Strain in Straps.

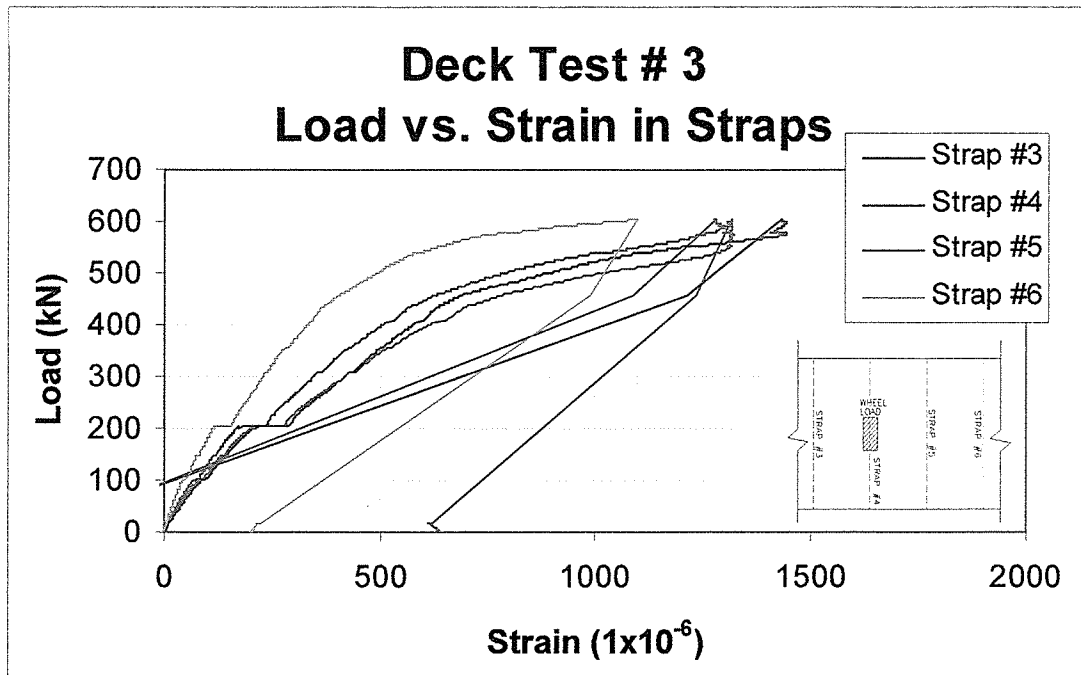


Figure 5-29. Deck Test #3 – Load vs. Strain in Straps.

5.3.3 LOAD VERSUS STRAIN IN TRANSVERSE GFRP BARS RESULTS

Load versus strain results in the transverse GFRP bars are summarised in Figures 5-30 and 5-31. The service wheel load of 87.5 kN and rupture strain of the GFRP bars has been shown on these figures. Below this load level no strain was measured in the GFRP bars. A small amount of strain was measured at 200 kN. An increase in strain was observed as the load increased beyond 200 kN. At failure the maximum strain in the GFRP bars was 9,000 μ strain which is well below the rupture strain of 18,600 μ strain.

It's important to note that under service loads the transverse GFRP bars have virtual no strain. Therefore, they provide no lateral restraint and have little effect on the capacity of the deck slab.

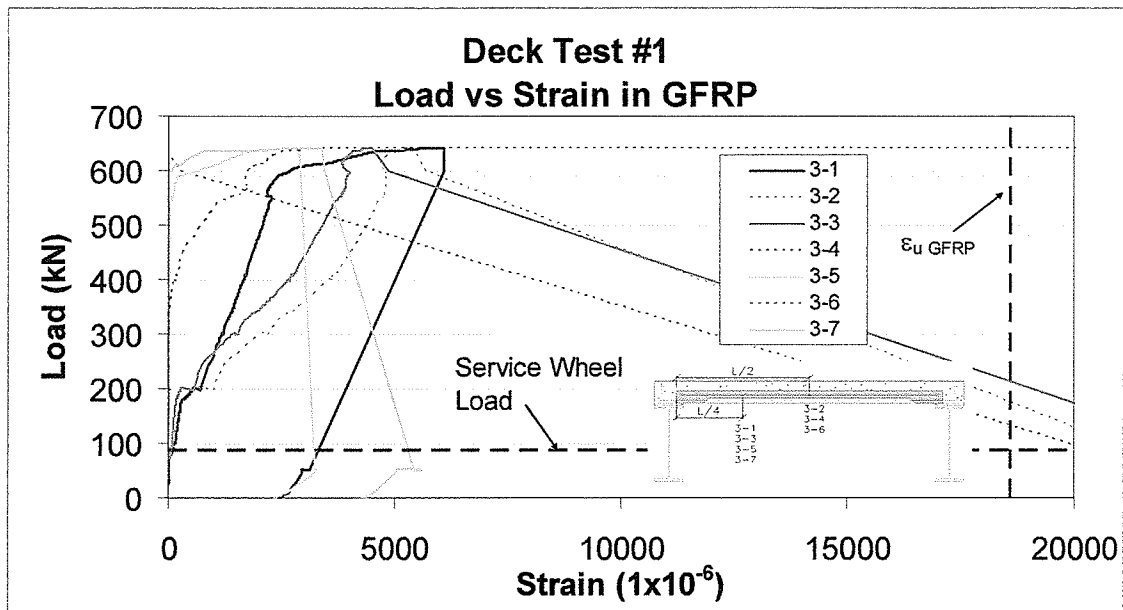


Figure 5-30. Deck Test #1 – Load versus Strain in Transverse GFRP Bars.

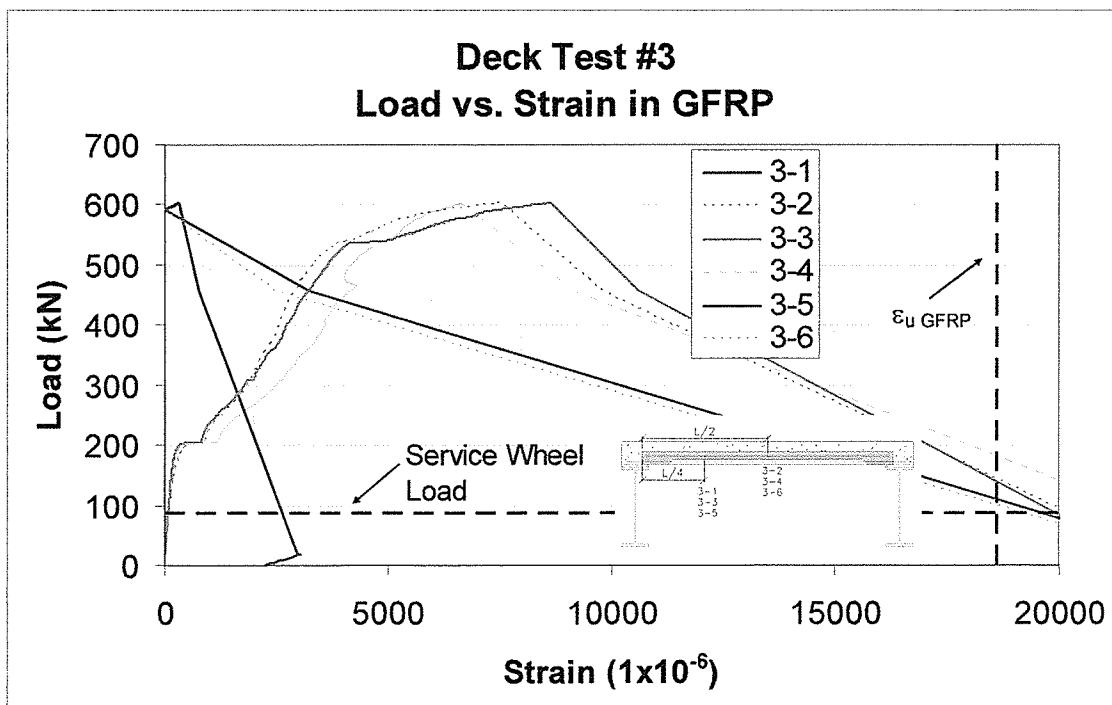


Figure 5-31. Deck Test #3 – Load versus Strain in Transverse GFRP Bars.

5.3.4 *LOAD VERSUS DEFLECTION RESULTS*

Load versus deflection results for tests one to three are summarised in Table 5-6 and shown in Figures 5-29 to 5-31. The deflections ranged from 18.0 mm to 32.8 mm at failure.

Smaller deflections were recorded in tests one and two which were at the ends of the deck specimen. Deflections for test three at the mid span of the deck specimen were approximately one and a half times those deflections. This may have occurred since the deck specimen was partially damaged after the first two tests.

All deflection measurements were taken at the midspan at approximately 50 mm on either side of the wheel load as shown in Figures 5-32 to 5-34.

Table 5-6. Deck Test Deflection at Failure Load.

Deck Test Number	Failure Load (kN)	Midspan Deflection at Failure (mm)
Test #1	642	LVDT East - 18.2 LVDT West - 18.0
Test #2	573	LVDT East - 20.1 LVDT West - 19.9
Test #3	603	LVDT East - 32.8 LVDT West - 32.1

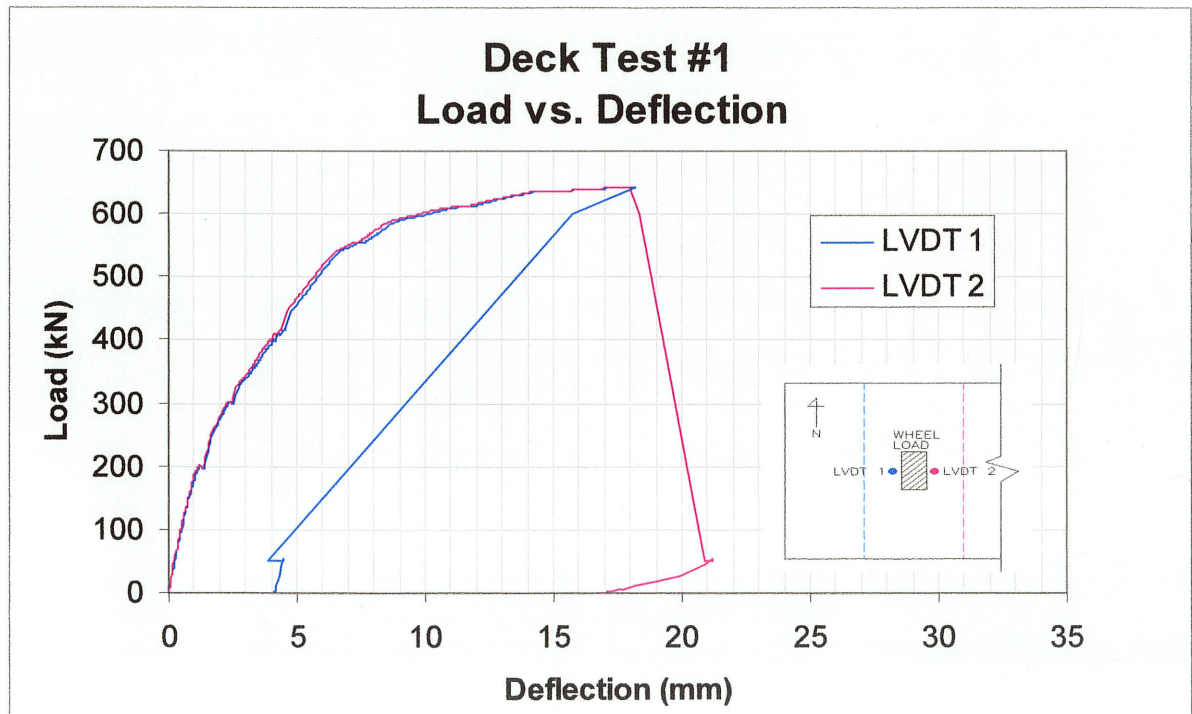


Figure 5-32. Deck Test #1 – Load versus Deflection.

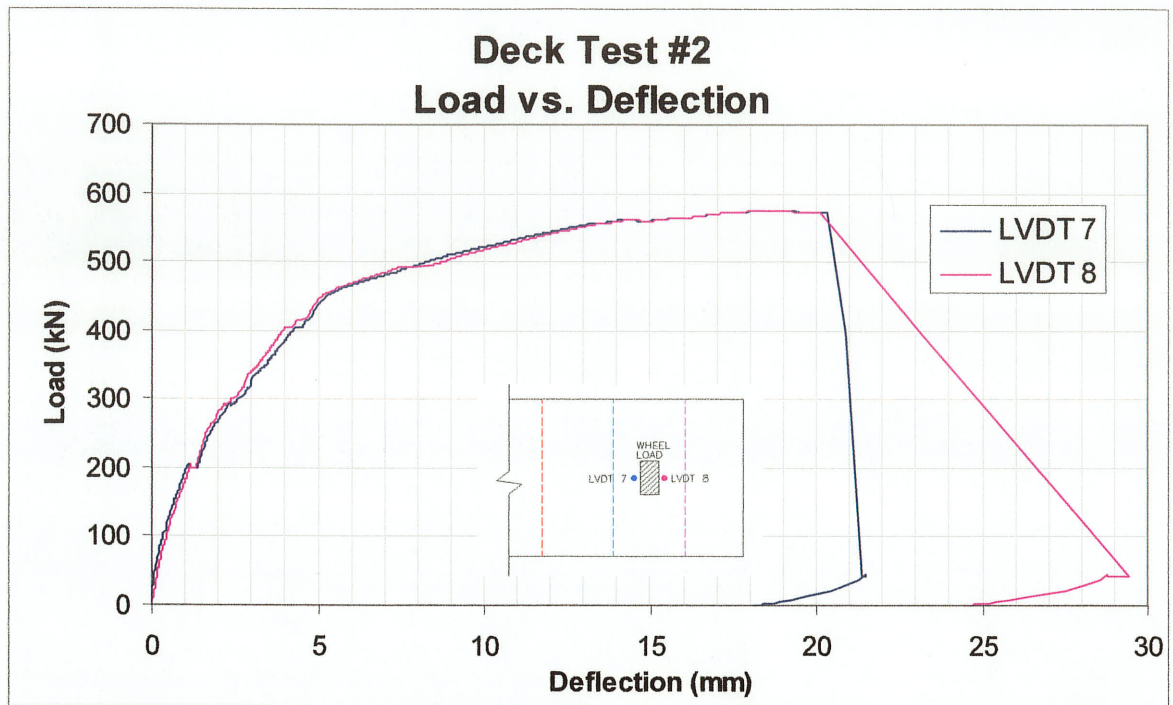


Figure 5-33. Deck Test #2 – Load versus Deflection.

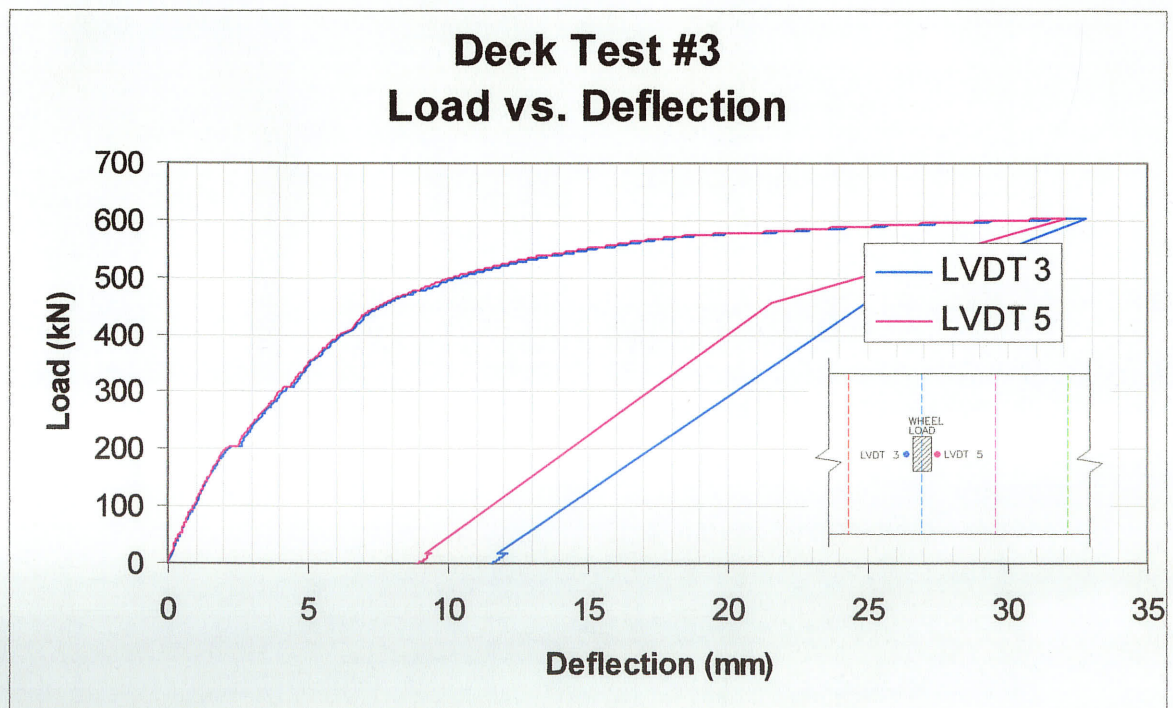


Figure 5-34. Deck Test #3 – Load versus Deflection.

5.3.5 *CONCRETE TESTING*

Both compression tests and tension tests were performed on the concrete during the testing day or within two days of the testing day. The results are summarised in Table 4-2.

5.3.6 *SPLITTING FORCES IN FLANGE*

One third of the deck was reinforced with hair pin bars over the flange. Six of these bars were instrumented with strain gauges. Load versus strain results from these gauges are shown in Figures 5-35 and 5-36. Strain values were less than 40 micro strain in all gauges prior to failure with the exception of the strain gauges located at the ends of the panel and deck specimen (strain gauges H3 and H6). A visible increase in strain was measured after approximately 400 kN and continued to increase until failure reaching strains of 350 to 600 micro strain on the north and south side of the test specimen.

The rupture strain of the concrete was calculated to be 130 micro strain. The largest strain in the rebar at a factored wheel load of 200 kN was 8.1 micro strain that was measured by gauge number V6. Based on this information, it is unlikely that the strain in the concrete will exceed the rupture concrete strain under static loads factored wheel loads.

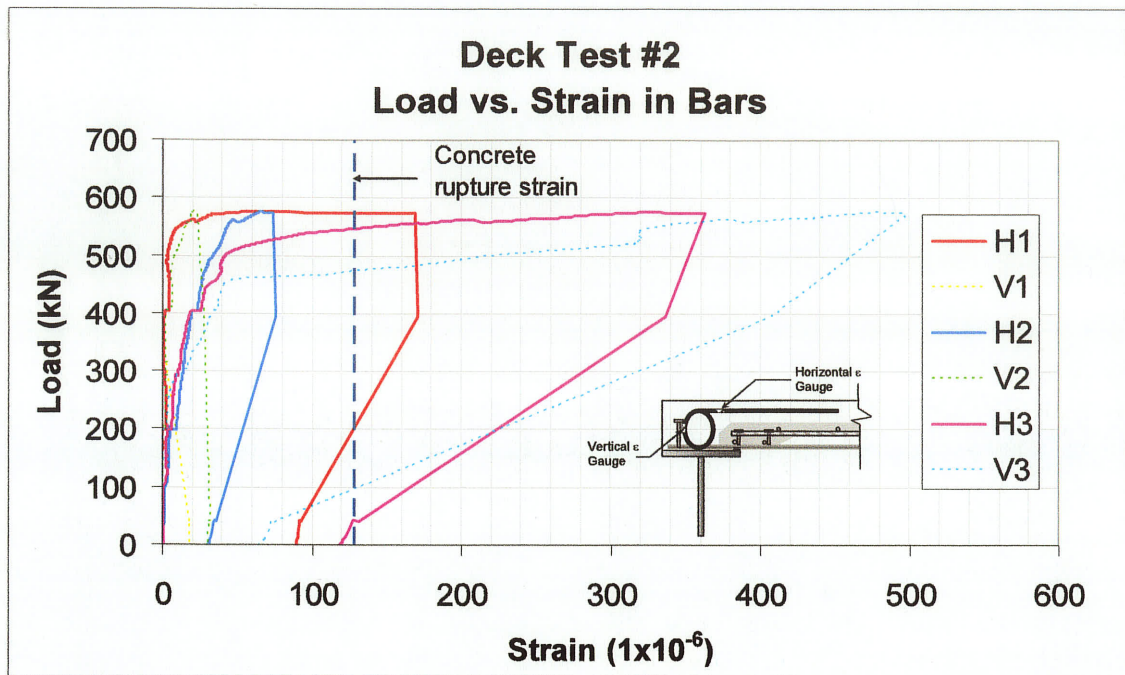


Figure 5-35. Deck Test #2 – Load versus Strain in Reinforcement Above Top Flange (North Side of Deck).

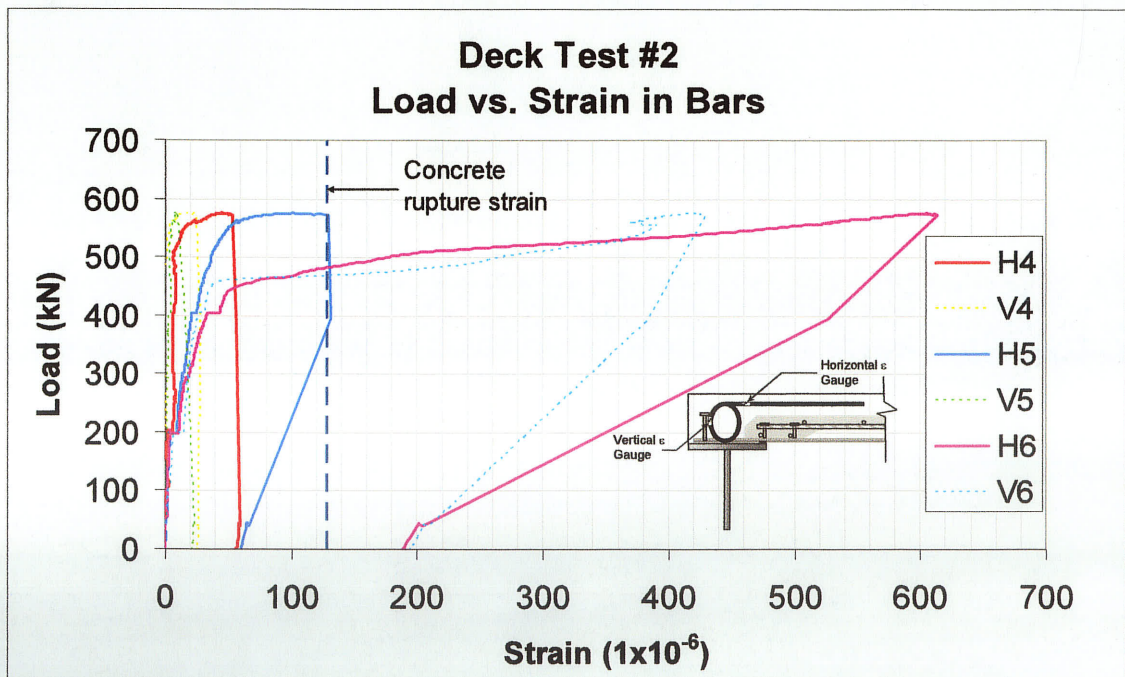


Figure 5-36. Deck Test #2 – Load versus Strain in Reinforcement Above Top Flange (South Side of Deck).

6.0 ANALYTICAL MODEL AND VERIFICATION WITH EXPERIMENTAL RESULTS

6.1 EXPERIMENTAL PHASE 2 – BRIDGE DECK

An analytical model was used to verify the experimental test results of the bridge deck or phase two of this testing program. The method of construction was neglected in the analytical model, assuming that it would act in a similar manner to a steel free deck cast monolithically. A rational analysis and a finite element analysis were carried out using programs titled "PUNCH" and "PUNCH FEM". The following describes the assumptions in the analysis and results.

6.1.1 *RATIONAL MODEL*

DESCRIPTION OF PUNCH PROGRAM

The PUNCH program was developed to predict the behaviour of laterally restrained concrete slab-on-girder bridge decks under wheel loading. This program requires a series of geometric input parameters to calculate the failure load of this type of deck and the corresponding strain in the steel straps.

A concrete slab on girder bridge deck that is constrained will eventually form a crack pattern shown in Figure 6-1 below. This crack pattern radiates from the applied load and is composed of a series of concrete wedges. Figure 6-2 shows an idealisation of the rotation of each of these wedges under loading. Provided that there is adequate lateral and longitudinal restraint the concrete will be under a three dimensional state of compression. Ultimately, it is the failure of the concrete in this region which leads to the failure of the slab by punching shear.

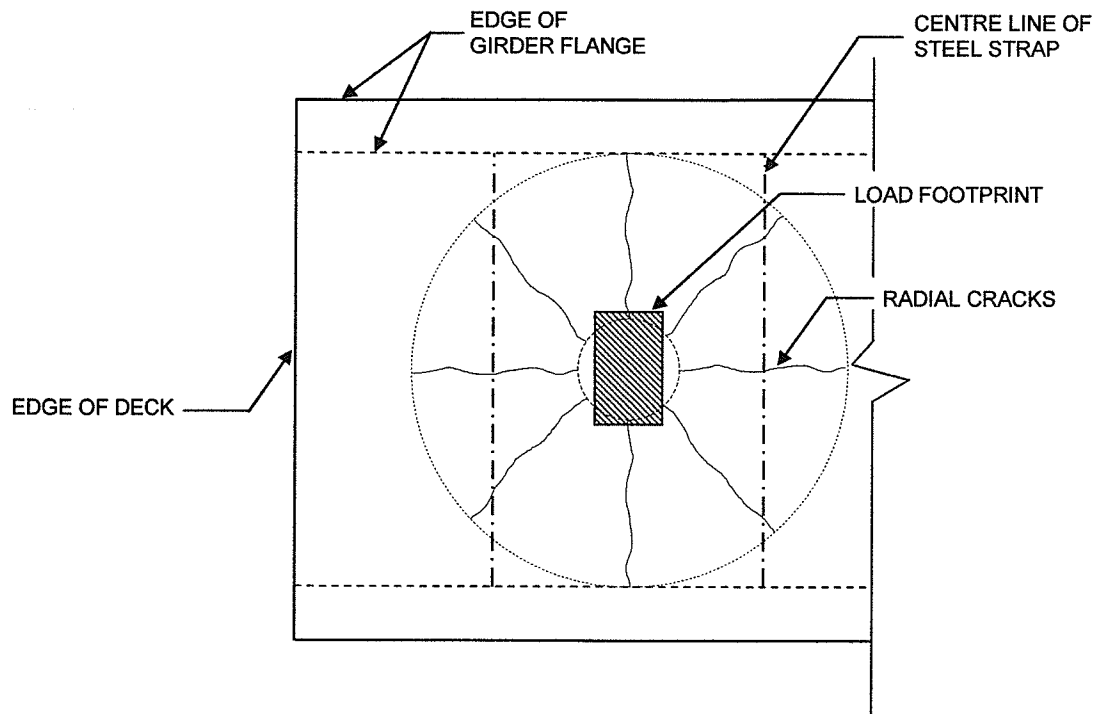


Figure 6-1. Punching Shear Radial Cracking Pattern at Underside of Deck at Failure.

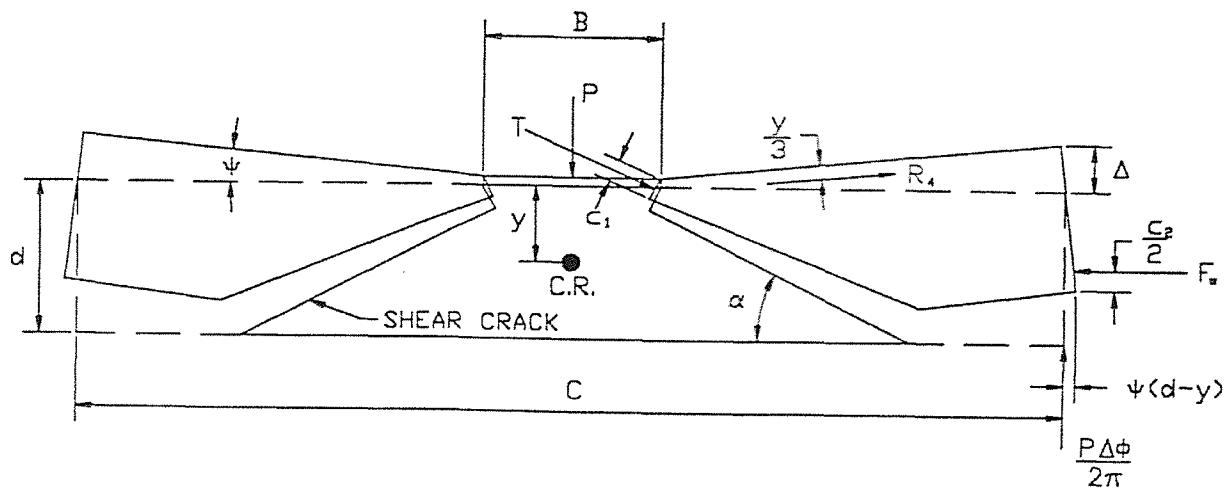


Figure 6-2. Theoretical Wedges Under Rigid Body Motion (Newhook, 1997).

The problem is non-linear and solved by an iterative process whereby the stresses and strains are calculated for each increment of slab deflection. The slab will fail by the concrete

crushing or the steel straps yielding which leads to subsequent crushing of the concrete. The equilibrium condition is compared to both of these criteria at each increment until one fails. The program can predict the slabs response for both single wheel loads and tandem wheel loads.

INPUT PARAMETERS

The input parameters required to run the program are described below and the inputs are provided:

S_l is defined as the length of the strap of a prismatic cross section, A_s , which is free to elongate. In the case where the strap is welded to the girder flange, this distance is taken as the clear distance between the girders. In our case, where the straps were anchored using shear connectors, the distance is defined as the length between the centres of anchorages. Therefore, a value of 2,550 mm was used in our analysis.

The program notes that the depth of the concrete deck should include the depth of the concrete plus the height of any haunch. This assumes that the punching failure will occur through the entire concrete cross section. The thickness of concrete at midspan was 175 mm and at the haunch was 250 mm. Both of these values were used in the analysis.

The transverse restraint is provided by the axial stiffness of the steel straps and the lateral stiffness of the supporting girder. Experimental analysis by Newhook has shown that the strap and girder stiffness can be idealised as a beam on elastic springs. In order to simplify the design, the PUNCH program conservatively estimates the lateral restraint of the steel straps by using the following equation:

$$K = \frac{2 E A_s}{S_l S_g} = \frac{200,000 \text{ MPa} \times 2,500 \text{ mm}^2}{1200 \text{ mm} \times 2550 \text{ mm}} = 327 \text{ MPa}$$

The user is also required to input a confinement constant which in the absence of additional information is ten (Newhook et al, 2003). This is required to model the three dimensional confinement state of the slab.

The analysis is based on a circular wheel load therefore the wheel footprint needs to be idealised as an equivalent circular area. This is done using the following equation:

$$B = 2(b+w)/\pi = 2(305 + 610)/\pi = 583 \text{ mm}$$

The following is a sequential list of all inputs used in the PUNCH program.

$$B = 583$$

$$f'_c = 43 \text{ MPa}$$

$$K = 327$$

$$D = 0$$

$$d = 175 \text{ mm}$$

$$B_1 = 0.85$$

$$K = 10$$

$$\Delta = \frac{b \times d \times 2}{\pi} = \frac{305 \text{ mm} \times 610 \text{ mm} \times 2}{\pi} = 118,503$$

$$\varepsilon_y = 0.002$$

RESULTS

The program calculated the load versus strap strain and deflection until failure of the slab using the inputs above. The results are summarised in Table 6-1 and shown in Figure 6-1.

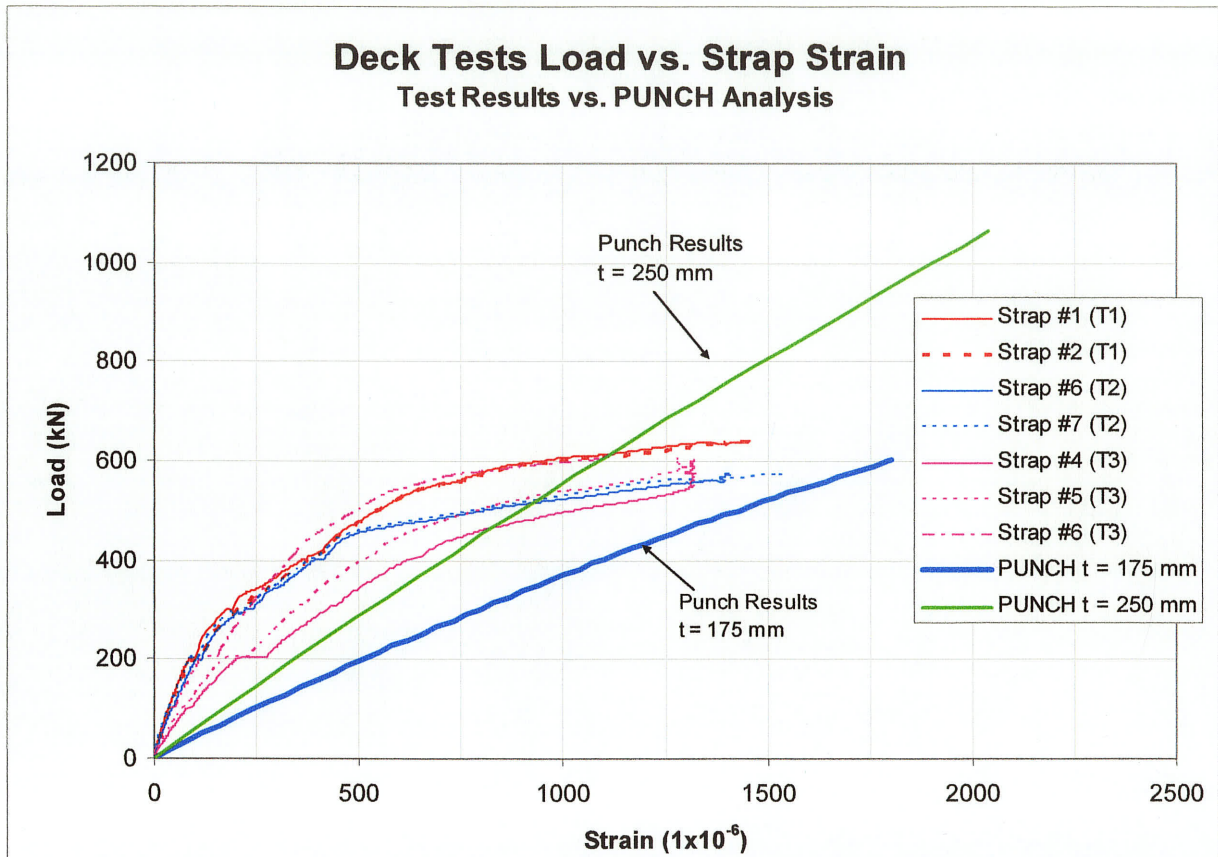


Figure 6-3. Comparison Test Data to PUNCH Data Load versus Strap Strain Results.

Table 6-1. Comparison of Test Data to PUNCH Data Strap Strains at Failure Load.

Test #	Description	Failure Load	Strap Strain (1×10^6)		
			West Strap	Centre Strap	East Strap
Test #1 West End of Deck	Load Centred Between Two Straps	632 kN	1436	N/A	1413
Test #2 East End of Deck	Load Centred Between Two Straps	573 kN	1408	N/A	1536
Test #3 Middle of Deck	Load Directly Above Strap	603 kN	1435	1320	1283
Punch Prediction (t = 175 mm)	-	602 kN	1800	-	-
Punch Prediction (t = 250 mm)	-	1065 kN	2040	-	-

Based on the results from analysis and testing, the PUNCH program accurately estimates the failure capacity of these panels when a 175 mm thick deck is analysed. The steel strap failure strain computed by the PUNCH program was approximately 30 % greater than those recorded during the testing.

6.1.2 FINITE ELEMENT ANALYSIS

DESCRIPTION OF PUNCH FEM

Similar to the PUNCH program, PUNCH FEM was developed to predict the response of concrete bridge decks that are laterally restrained and loaded under heavy truck wheel loading. It was designed specifically for the steel free deck and is based on a nonlinear finite element analysis encompassing a three dimensional state of stress. The program provides load versus deflection output at any location that the user inputs.

Cracking of the concrete is modeled through an adjustment in the material properties and by approximating the cracking as a smeared band of cracks. In addition, the stiffness of the steel straps is smeared across the length of the girders and lateral stiffness of the girders has been assumed constant over the length of the girders. The girders are unyielding in the vertical direction and the concrete is modeled as an orthotropic material. Both tensile and crushing failure of the concrete occurs along the principal direction.

Both single wheel loads and double axle loads can be analysed. The following is a summary of the program assumptions:

- The transverse restraint from the steel straps is smeared along the length of the girder.
- The lateral stiffness of the girders has been considered to be uniform across the length of the girder.
- The girders are unyielding in the vertical direction.
- Concrete is modelled as an orthotropic material with the direction of orthotropy being defined by the principal stress directions.
- Cracking can occur in three orthogonal directions at an integration point used in the formation of finite element directions.
- If cracking occurs at an integration point, it is modeled through an adjustment of material properties by considering cracking as a smeared band of cracks.
- Strain softening occurs from compression crushing failure to the ultimate strain, at which the concrete fails complete.
- The tensile cracking failure and the concrete crushing failures occur along the principal direction.
- Load increases monotonically, leading to failure.

INPUT PARAMETERS

The deck was modelled as having a thickness of 250 mm and 175 mm of concrete under constant depth. A concrete strength equal to 43 MPa was used in the analysis. Spring constants used to simulate the steel straps and the girders were:

If the total stiffness of seven straps is distributed evenly over the 9100 mm girder length then distributed stiffness due to the straps is calculated as strapx below.

$$\text{Strapx} = \frac{7 \text{ straps} \times 1250 \text{ mm}^2 \times 200,000 \text{ MPa}}{2,550 \text{ mm} \times 121} = 5,671 \text{ N/mm}$$

$$\begin{aligned} \text{Sprx} &= \text{Girder Cross Sectional Area} \times E / \text{Centre to centre distance between girders} \\ &= \frac{150 \text{ mm}^2 \times 200,000 \text{ MPa}}{2550 \text{ mm}} = 11,765 \text{ N/mm} + 5,671 \text{ N/mm} = 17,436 \text{ N/mm} \end{aligned}$$

The lateral spring constant was rounded to 20,000 N/mm during the analysis and varied to 30,000 N/mm. Increasing the spring constant to 30,000 N/mm did not change the ultimate load but reduced the deflection at the ultimate load by approximately 10%.

RESULTS

Results from PUNCH FEM are summarised below in Table 6-2 and shown below in Figures 6-2 and 6-3 which present the load versus deflection curves. In addition, results from the PUNCH program are included to allow a comparison of results between the two programs. The failure load of the FEM model with a 175 mm thick slab was 540 kN at a deflection of 9.4 mm when a spring constant of 30,000 N/mm was used in the analysis. The compressive strength of the concrete was 43 MPa.

Table 6-2. Summary of Slab Deflection at Failure of PUNCH FEM, PUNCH and Test Results

Test #	Description	Failure Load	Deflection (mm)	
			East LVDT	West LVDT
Test #1 West End of Deck	Load Centred Between Two Straps	632 kN	18.2	18.0
Test #2 East End of Deck	Load Centred Between Two Straps	573 kN	20.1	19.9
Test #3 Middle of Deck	Load Directly Above Strap	603 kN	32.8	32.1
PUNCH FEM	t = 175 mm	540	9.4	
PUNCH FEM	t = 250 mm	900	5.3	
PUNCH	t = 175 mm	602	26.2	
PUNCH	t = 250 mm	1065	19.3	

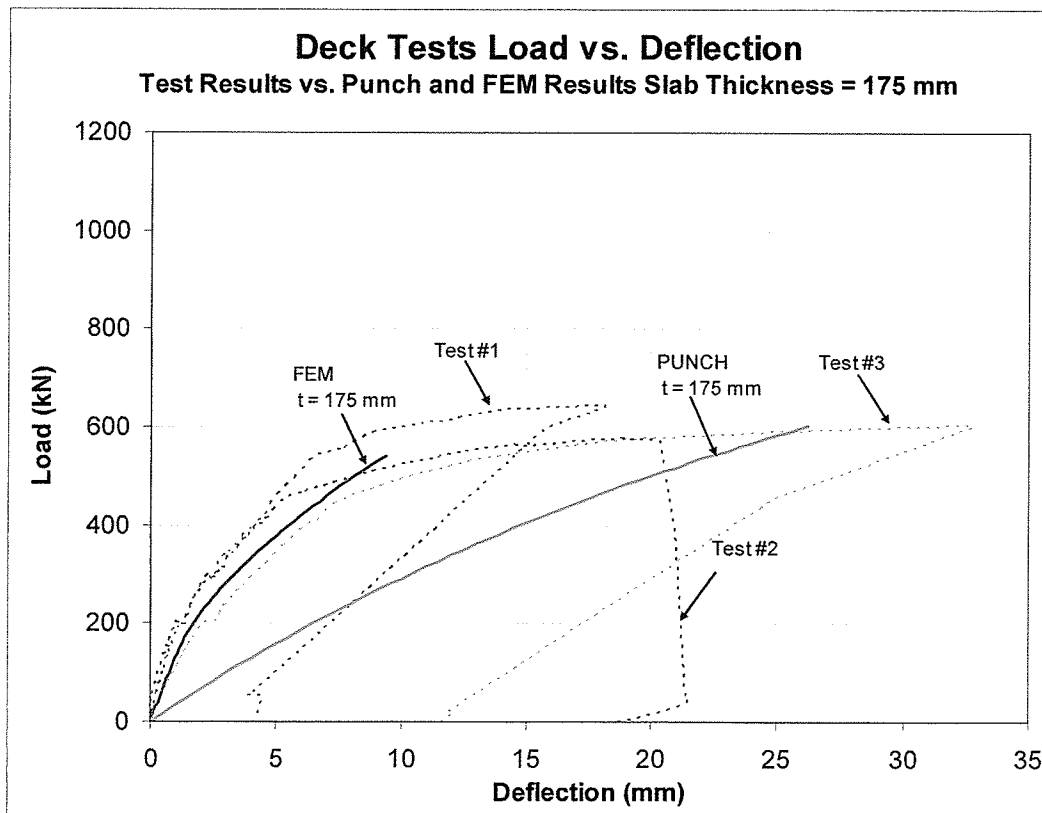


Figure 6-4. Comparison of Deck Test Load Versus Deflection Results to PUNCH and PUNCH FEM Results for a Deck Thickness Equal to 175 mm.

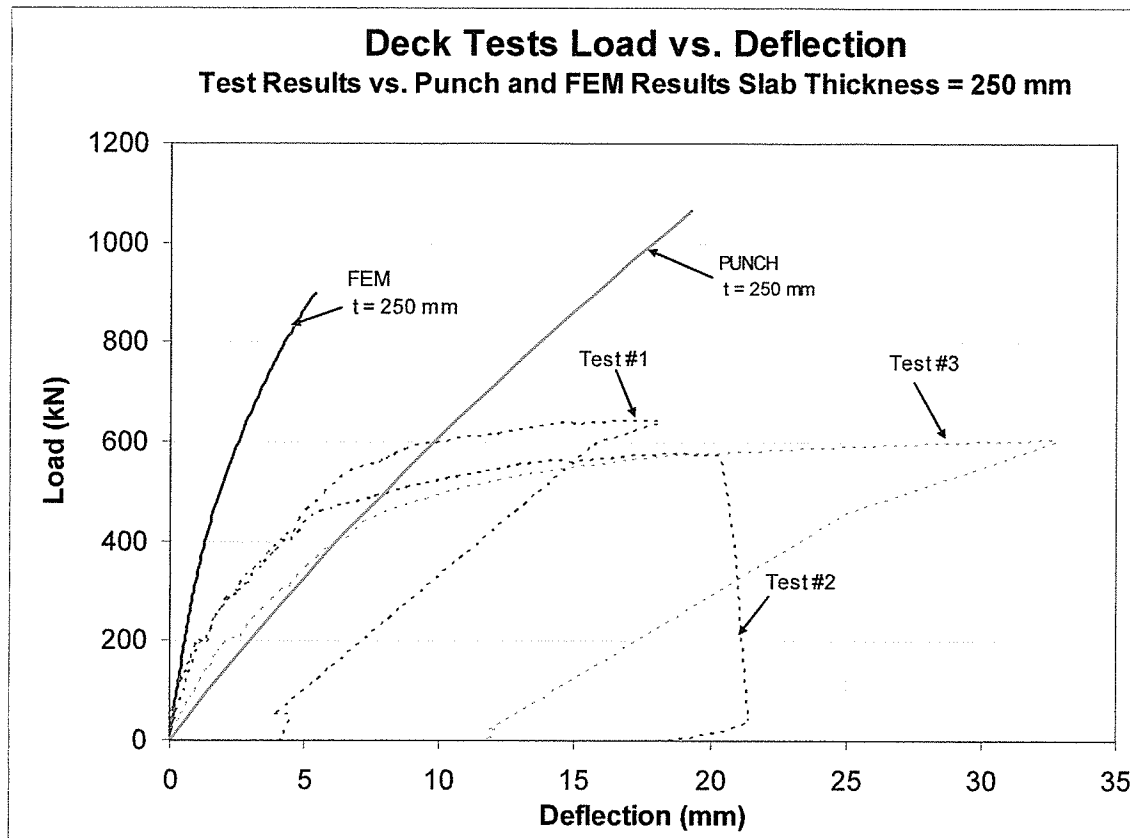


Figure 6-5. Comparison of Deck Test Load Versus Deflection Results to PUNCH and PUNCH FEM Results for a Deck Thickness Equal to 250 mm.

6.2 COMPARISON WITH RESULTS

The PUNCH FEM results provided a more accurate representation of the load deflection curve compare with the PUNCH program. In all cases the predicted deflection at failure was more than half the actual failure observed during the testing. Failure loads calculated using the PUNCH and PUNCH FEM programs provided a good correlation to the failure loads observed during the test when a concrete thickness equal to 175 mm was inputted.

7.0 CONCLUSION AND RECOMMENDATIONS

This testing program confirmed that a steel free deck constructed using partial depth precast concrete panels fails in punching shear similar to a monolithically cast steel free deck. The ultimate capacity of the deck compared well to the PUNCH and PUNCH FEM analytical models when a concrete thickness of 175 mm was analysed. The steel straps laterally restrained the supporting girders during loading. The strain in the transverse GFRP bars was negligible under service wheel loading and did not contribute to the ultimate capacity of the deck.

In addition, the testing program verified that the ArchForm partial depth precast concrete panels can safely withstand transportation and construction loading for the span arrangement utilised in this test. The ArchForm partial depth precast concrete panels resisted load primary in bending and the steel straps resisted the majority of the load not the GFRP bars cast within the panels. The grid of GFRP reinforcement cast within the deck provided crack control during the panel and deck testing.

The following recommendations apply to the use of partial depth precast concrete panels with the steel free deck system:

- Ensure that bearing detail utilised in testing program for partial depth precast panels is strictly followed in field applications.
- At the end panels, extend the longitudinal reinforcement into the thickened end portion of the deck slab.
- Provide a top layer of crack control reinforcement where the transportation authority recommends its use to mitigate reflective cracking from the panel joints.

- Provide fibre reinforced concrete in the upper lift concrete when there is no reinforcement in this concrete.
- Ensure that the width of the girder flange is 350 mm or greater to avoid construction difficulties during panel erection.

Additional study in the following areas is recommended:

- Perform fatigue testing under a rolling wheel load to determine the effects on the potential reflective cracking in the upper deck concrete at the transverse and longitudinal joints.
- Investigate the use of alternative crack control reinforcement grids in the cast in place concrete over the partial depth precast panel butt joints to eliminate the possibility of potential reflective cracking.
- Design and develop details for a cantilever panel section.
- Investigate long term performance of concrete deck at cracks on the deck surface for deck slabs without top reinforcement. Perform testing under varying temperatures and under load.

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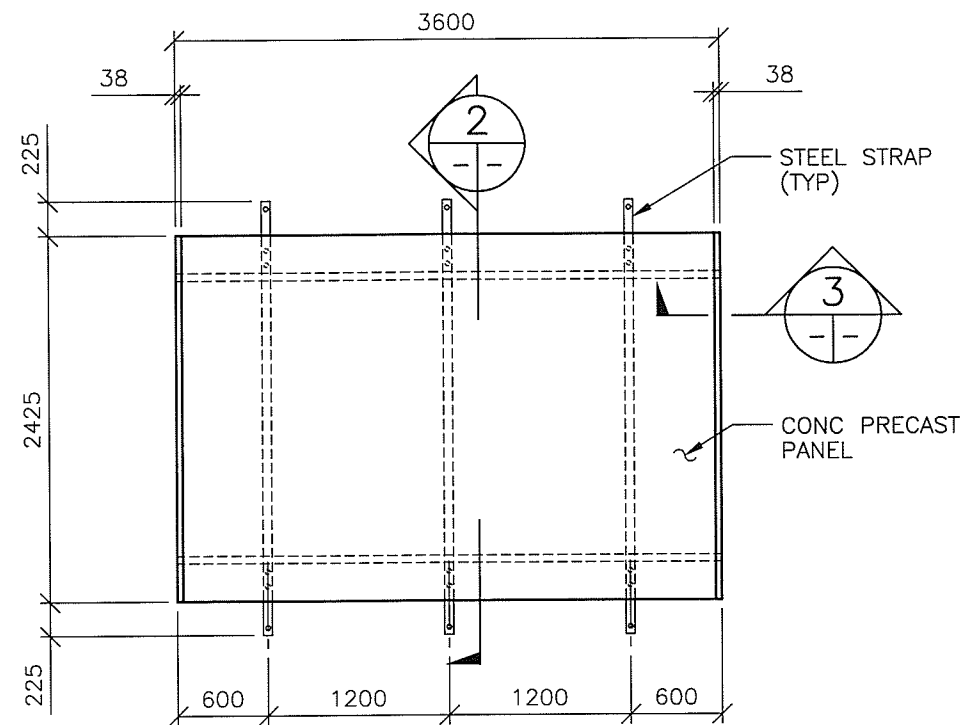
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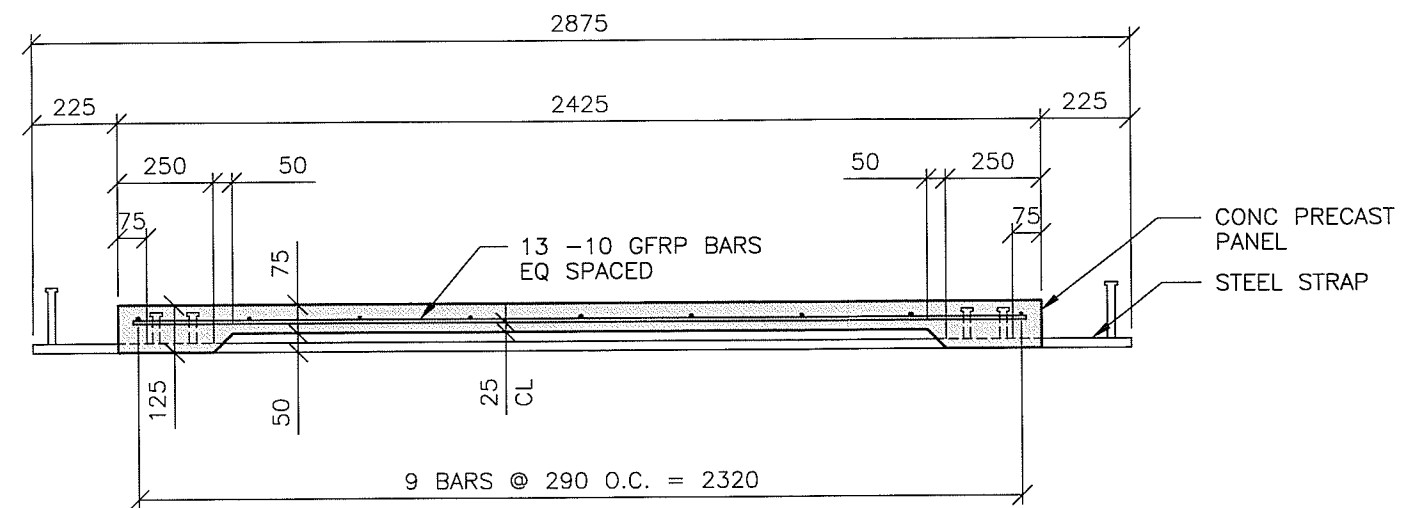
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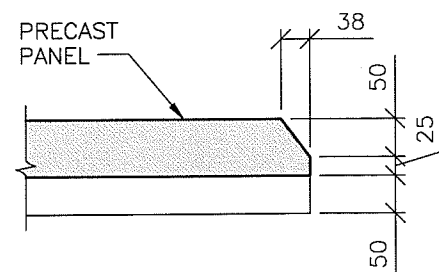
APPENDIX 1 – PANEL FABRICATION DRAWINGS FOR PHASE 1 AND 2 TESTING



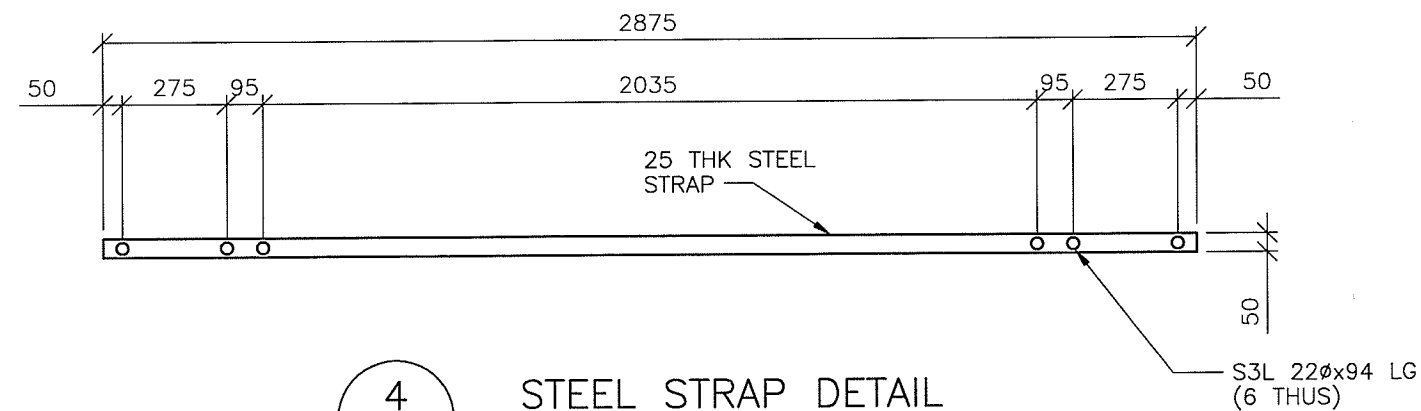
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- 3 REQUIRED



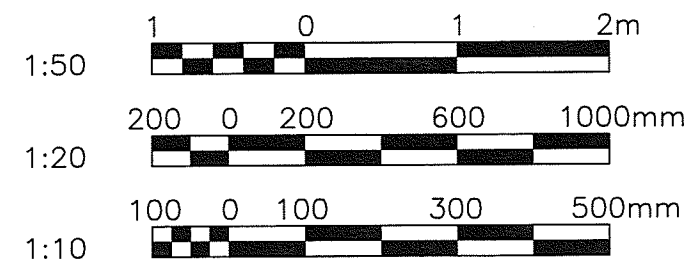
2 PANEL SECTION
1 : 20



3 SECTION
1 : 10

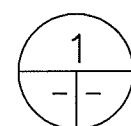
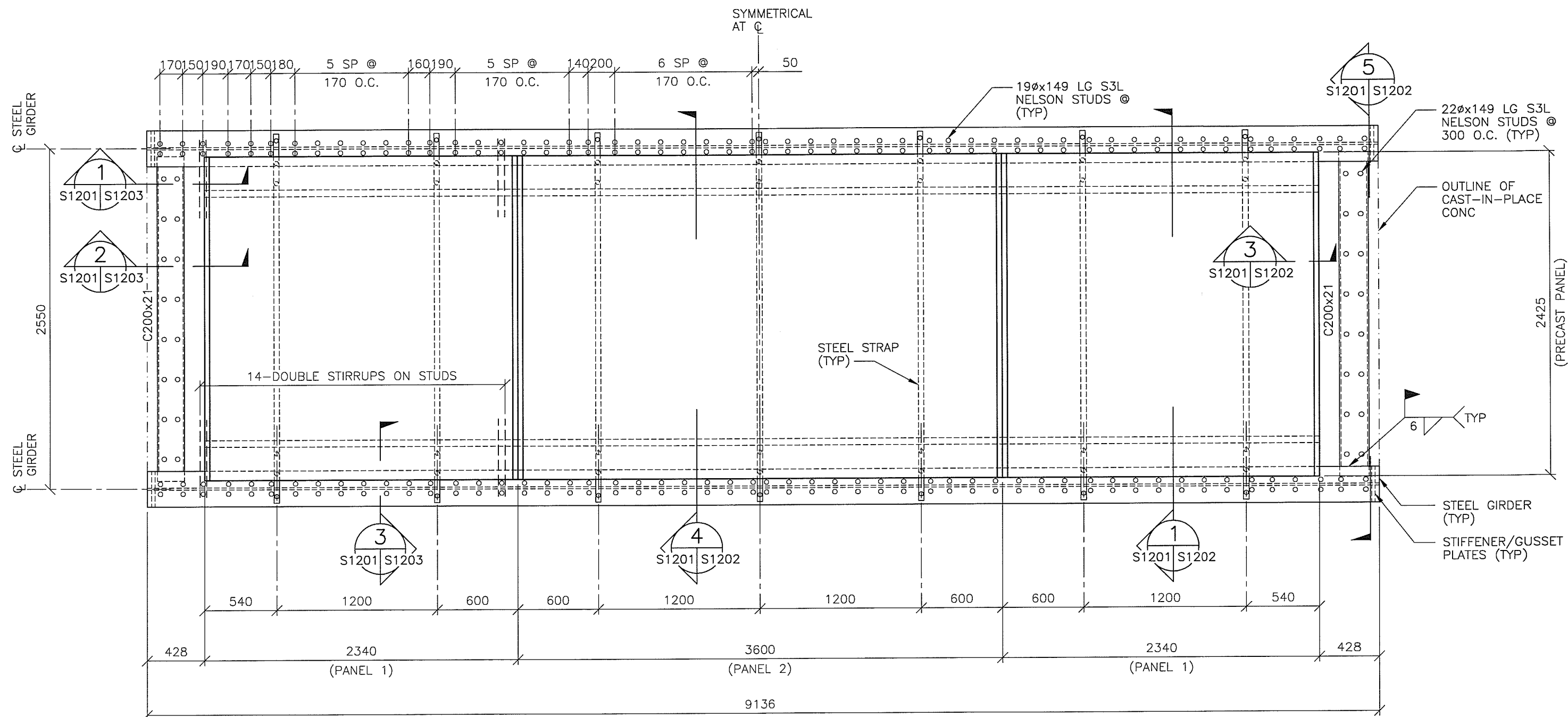


4 STEEL STRAP DETAIL
1 : 20



CONFIDENTIAL
PATENT PENDING

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DESIGNED BY: J.D.B.	DRAWN BY: G.I.	DWG NO.
CHECKED BY: J.D.B.	DATE: 04.02.03	0405000700-SKT-S1101



PRECAST PANEL LAYOUT

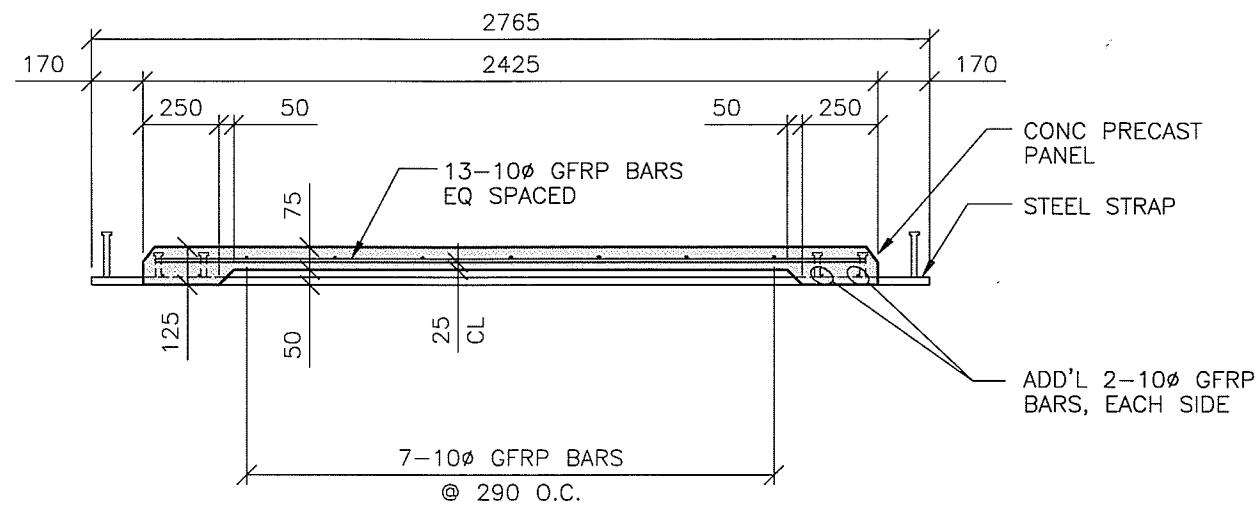
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— CAST IN PLACE CONC NOT SHOWN FOR CLARITY

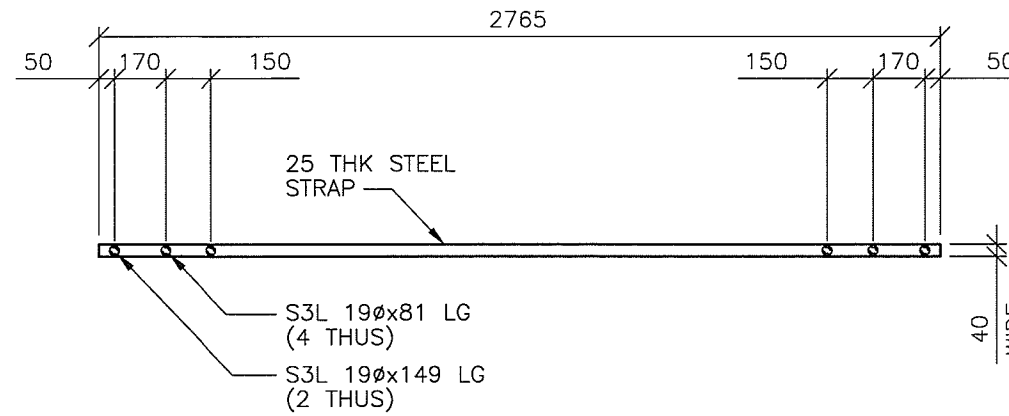
CONFIDENTIAL
PATENT PENDING

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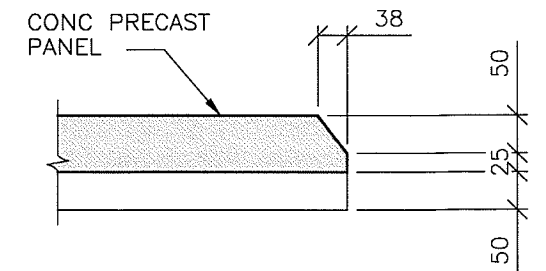
DWG DESCRIPTION		
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CHECKED BY: J.D.B.	DATE: 04.02.03	0405000700-SKT-S1201



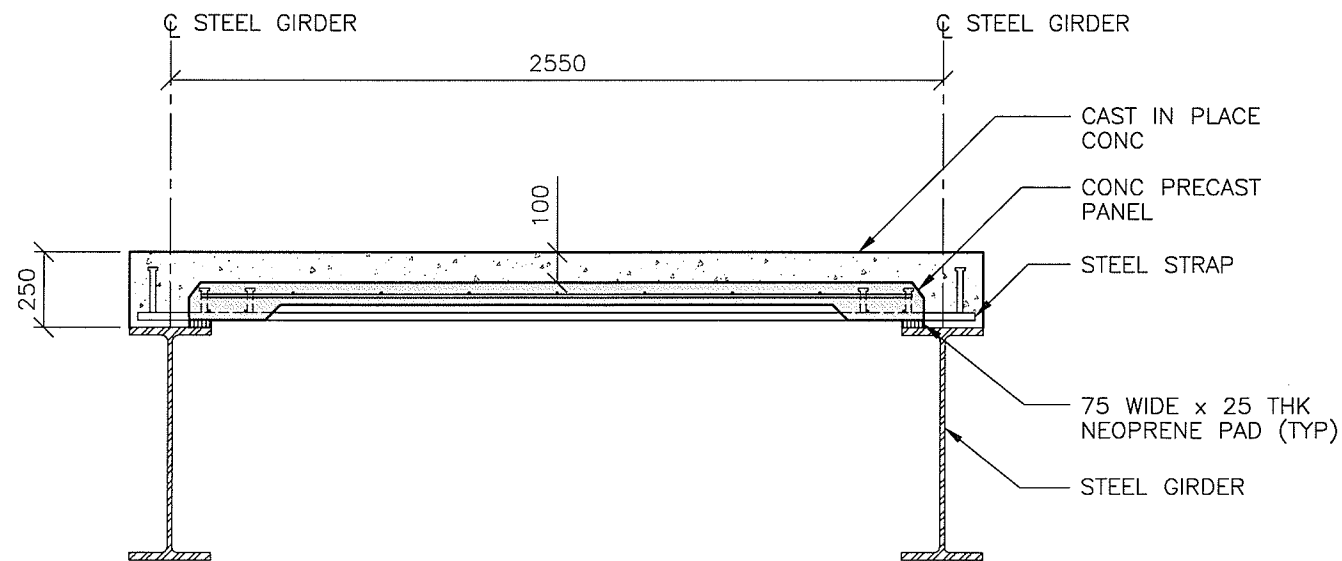
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S1201 | S1202 1 : 25



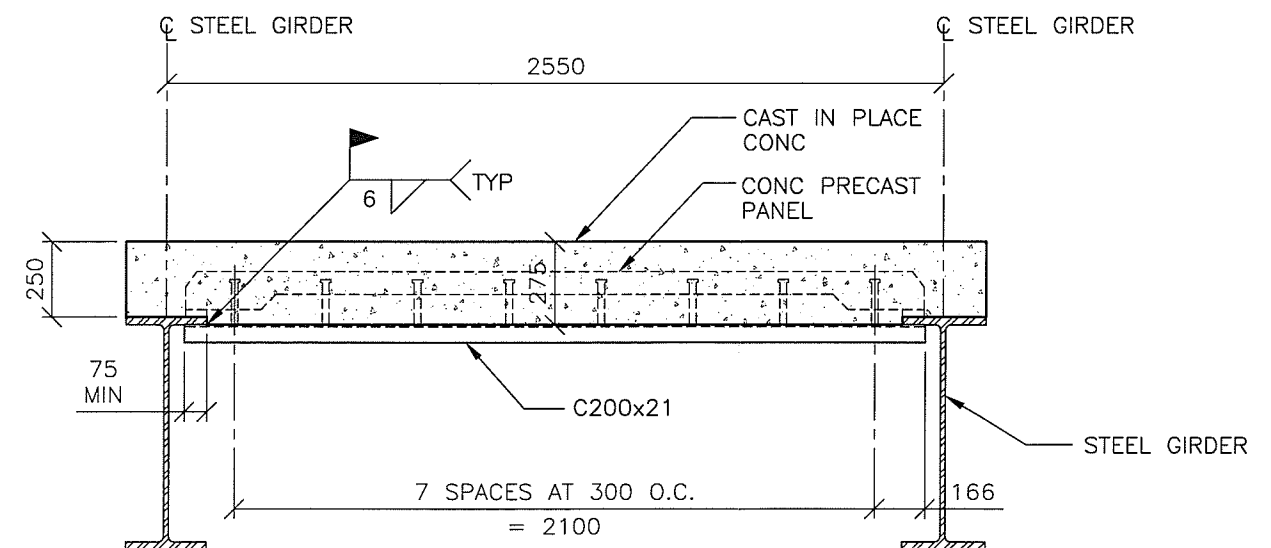
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S1201 | S1202 1 : 25



3 SECTION
S1201 | S1202 1 : 10

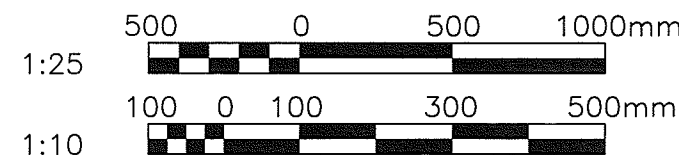


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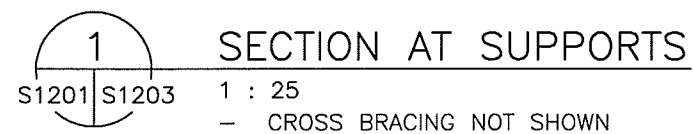


5 SECTION
S1201 | S1202 1 : 25

CONFIDENTIAL
PATENT PENDING



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CONFIDENTIAL
PATENT PENDING



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APPENDIX 2 – RESULTS FROM PUNCH ANALYSIS

PUNCH

UNITS METRIC KN, MM, SECOND

Clear Span Between Girders= 2550.00
Diameter of Equivalent Circle for Load= 582.000
Maximum Compressive Stress of Concrete= 43.0000
Elastic Axial Stiffness of Strap= 179.000
Strap to Load Spacing= 600.000
Depth of Slab= 175.000
Beta to Define Rectangular Stress Block= 0.850000
Concrete Constant used for confinement= 10.0000
Area of Load Patch= 186050.
Yield strain= 0.200000E-02

DELTA	ALPHA	P	STRN
0.00	0.00	0.	0.00000
0.50	15.79	17.	0.00004
1.00	15.64	34.	0.00008
1.50	15.49	51.	0.00012
2.00	15.35	67.	0.00016
2.50	15.22	83.	0.00020
3.00	15.09	99.	0.00024
3.50	14.97	114.	0.00028
4.00	14.85	129.	0.00032
4.50	14.74	144.	0.00036
5.00	14.63	159.	0.00040
5.50	14.53	173.	0.00044
6.00	14.43	187.	0.00047
6.50	14.33	201.	0.00051
7.00	14.24	214.	0.00055
7.50	14.15	228.	0.00058
8.00	14.07	241.	0.00062
8.50	13.98	254.	0.00066
9.00	13.90	266.	0.00069
9.50	13.83	279.	0.00073
10.00	13.75	291.	0.00076
10.50	13.68	303.	0.00080
11.00	13.61	315.	0.00083
11.50	13.54	327.	0.00087
12.00	13.47	339.	0.00090
12.50	13.41	350.	0.00094
13.00	13.34	361.	0.00097
13.50	13.28	372.	0.00100
14.00	13.22	383.	0.00104
14.50	13.16	394.	0.00107
15.00	13.10	404.	0.00110
15.50	13.05	415.	0.00114
16.00	12.99	425.	0.00117
16.50	12.94	435.	0.00120
17.00	12.89	445.	0.00123
17.50	12.84	454.	0.00127
18.00	12.79	464.	0.00130
18.50	12.74	474.	0.00133
19.00	12.69	483.	0.00136
19.50	12.64	492.	0.00139
20.00	12.60	501.	0.00143
20.50	12.55	510.	0.00146
21.00	12.51	519.	0.00149
21.50	12.47	527.	0.00152
22.00	12.42	536.	0.00155
22.50	12.38	544.	0.00158
23.00	12.34	552.	0.00161
23.50	12.30	560.	0.00164
24.00	12.26	568.	0.00167
24.50	12.22	576.	0.00170
25.00	12.19	584.	0.00173

PUNCH

25.50	12.15	591.	0.00176
26.00	12.11	599.	0.00179
26.22	12.10	602.	0.00180

***** Punch Load-Strain in concrete=.002 *****

PUNCH

UNITS METRIC KN, MM, SECOND

Clear Span Between Girders= 2550.00
Diameter of Equivalent Circle for Load= 582.000
Maximum Compressive Stress of Concrete= 43.0000
Elastic Axial Stiffness of Strap= 179.000
Strap to Load Spacing= 600.000
Depth of Slab= 250.000
Beta to Define Rectangular Stress Block= 0.850000
Concrete Constant used for confinement= 10.0000
Area of Load Patch= 186050.
Yield strain= 0.200000E-02

DELTA	ALPHA	P	STRN
0.00	0.00	0.	0.000000
0.71	21.69	50.	0.000008
1.43	21.23	99.	0.000017
2.14	20.82	147.	0.000025
2.86	20.46	193.	0.000033
3.57	20.12	239.	0.000041
4.29	19.82	283.	0.000049
5.00	19.55	327.	0.000057
5.71	19.29	370.	0.000065
6.43	19.06	412.	0.000073
7.14	18.84	453.	0.000080
7.86	18.64	494.	0.000088
8.57	18.45	534.	0.000096
9.29	18.27	573.	0.000103
10.00	18.10	611.	0.000111
10.71	17.94	649.	0.000118
11.43	17.79	687.	0.000125
12.14	17.65	723.	0.000133
12.86	17.52	760.	0.000140
13.57	17.39	795.	0.000147
14.29	17.27	830.	0.000155
15.00	17.15	865.	0.000162
15.71	17.04	899.	0.000169
16.43	16.93	933.	0.000176
17.14	16.83	966.	0.000183
17.86	16.73	998.	0.000190
18.57	16.63	1031.	0.000197
19.29	16.54	1065.	0.000204

***** Punch Load-Yield Strain in Strap *****