1-1-2010

Push-Off Tests On Full-Depth Precast Concrete Bridge Deck Panel To Steel Girder Connection

Bahman Marvi
Ryerson University

PUSH-OFF TESTS ON FULL-DEPTH PRECAST CONCRETE BRIDGE DECK PANEL TO STEEL GIRDER CONNECTION

By

Bahman Marvi, M.Eng.

Ryerson University, 2010

A Project Report

Presented to Ryerson University

In partial fulfillment of the

Requirements for the degree of

Master of Engineering in the Program of

Civil Engineering

Toronto, Ontario, Canada, 2010

AUTHOR'S DECLARATION

I hereby declare that I am the sole author of this thesis.

I authorize Ryerson University to lend this thesis to other institutions or individuals for the purpose of scholarly research.

Author's Signature.......................................   Date...........................

I further authorize Ryerson University to reproduce this thesis by photocopying or by other means, in total or in part, at the request of other institutions or individuals for the purpose of scholarly research.

Author's Signature.......................................   Date...........................

BORROWER'S PAGE

Ryerson University requires the signatures of all persons using or photocopying this thesis.

Please sign below, and give address and date.

Name	Signature of Borrower	Address	Date

ABSTRACT

PUSH-OFF TESTS ON FULL-DEPTH PRECAST CONCRETE BRIDGE DECK PANEL TO STEEL GIRDER CONNECTION

By

Bahman Marvi

Master of Engineering in Civil Engineering

Department of Civil Engineering, Ryerson University

Toronto, Ontario, Canada

2010

The use of prefabricated elements and systems in bridge construction has recently been the subject of much attention and interest amongst bridge jurisdictions as a way of improving bridge construction. Through mass production of the materials, the repeated use of forms, reduction of on-site construction time and labour by concentrating the construction effort in a fabrication facility rather than at the bridge site, significant economic benefits can be achieved. Aging bridges of North America may require repair, rehabilitation, or replacement. The current traditional bridge rehabilitation/replacement system in most situation is very time consuming and costly. Issues related to work zone safety and traffic disruptions are also a major concern. A full-lane closure is very costly in large busy urban highways because of the significant economic impact on commercial and industrial activities. As a result, prefabricated bridge technology is seen as a potential solution to many of these issues. Prefabricated elements and systems can be quickly assembled and could reduce design efforts, reduce the impact on the environment in the vicinity of the site, and minimize the delays and lane closure time and inconvenience to the traveling public, saving time and tax payers' money. This project investigates the full-depth precast bridge deck panels with no overlays connected to the steel girders using lumped shear connectors Eight panel-steel girder connections of different shear connector configurations were erected and tested to complete collapse to examine their structural behaviour, crack pattern and ultimate load carrying capacity. Based on the experimental findings, recommendations for practical applications were drawn.

ACKNOWLEDGEMENTS

The author would like to express my deep thanks to his supervisor, Dr. K. Sennah, whose constant support, excellent guidance, supervision and scholarly counsel that has allowed this project to carry forward and fulfill expectation

. The author wishes to thank Dr. Clifford Lam and Mr. Ben Huh from Ministry of Transportation of Ontario (MTO) for his valuable suggestions, assistance and enthusiasm that were essential pieces of this research. Special thanks to the MTO Bridge Office for supplying the test specimens.

The author would like to thank Mr. Nidal Jaalouk for his continuous support, commitment and dedication was an integral part of this project. This support is greatly appreciated.

TABLE OF CONTENTS

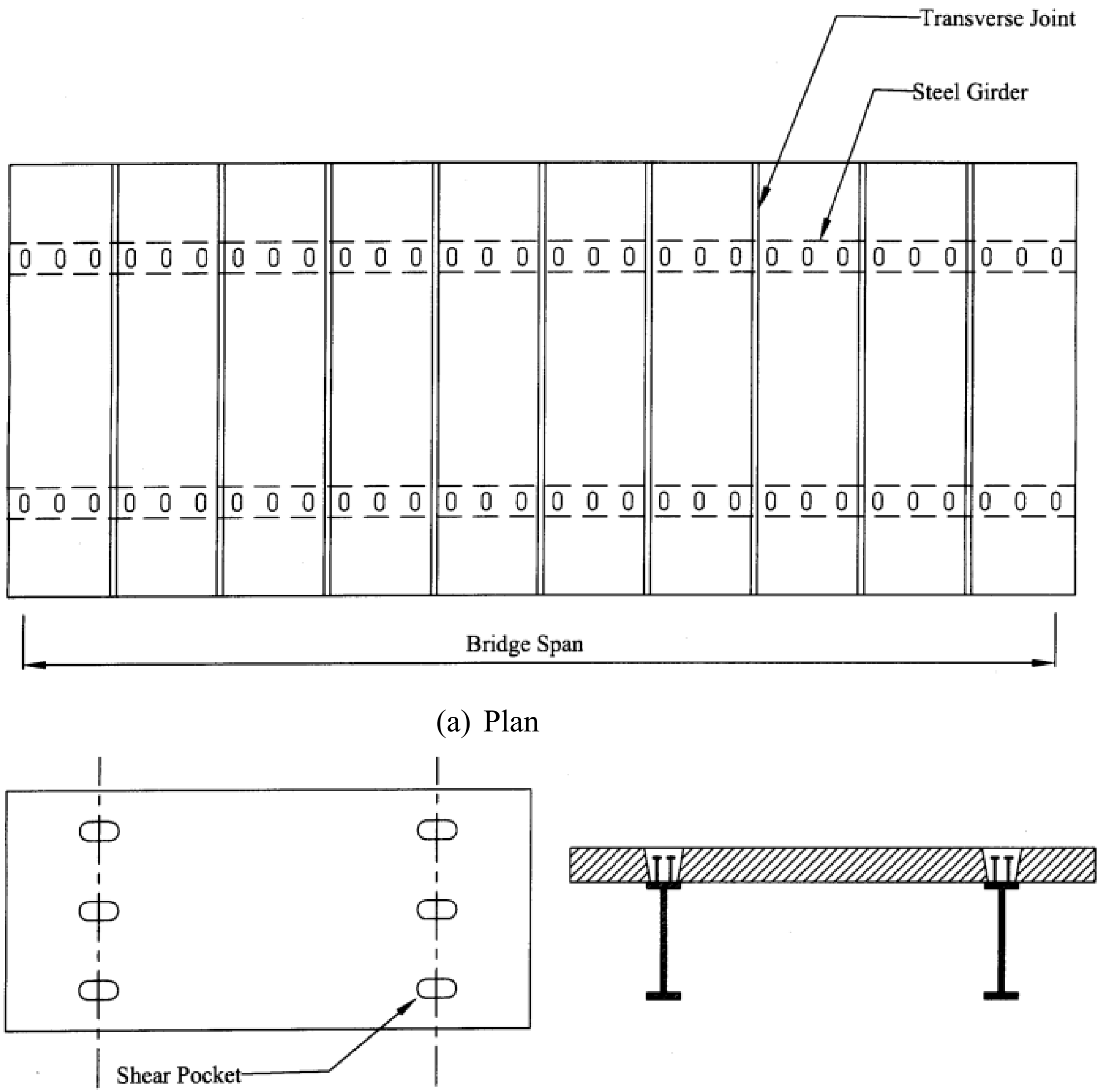
CHAPTER I
INTRODUCTION

1.1. Introduction

The use of prefabricated elements and systems in bridge construction has recently been the subject of much attention and interest amongst bridge jurisdictions as a way of improving bridge construction. Through mass production of the materials, the repeated use of forms, reduction of on-site construction time and labour by concentrating the construction effort in a fabrication facility rather than at the bridge site, significant economic benefits can be achieved. Aging bridges of North America may require repair, rehabilitation, or replacement. The current traditional bridge rehabilitation/replacement system in most situation is very time consuming and costly. Issues related to work zone safety and traffic disruptions are also a major concern. A full-lane closure is very costly in large busy urban highways because of the significant economic impact on commercial and industrial activities. As a result, prefabricated bridge technology is seen as a potential solution to many of these issues. Prefabricated elements and systems can be quickly assembled and could reduce design efforts, reduce the impact on the environment in the vicinity of the site, and minimize the delays and lane closure time and inconvenience to the traveling public, saving time and tax payers' money. Even at a higher initial cost, the use of prefabricated systems on bridges subjected to a high volume of traffic may be justified, because excessive lane closure times can be avoided. This technology is applicable and needed for both existing and new bridge construction.

1.2. The Problem

One of the attractive solutions to accelerate the construction of new bridges or replacement of deteriorated ones is the use of full-width, full-depth precast deck panel design in which the panels cover part of the entire width of the bridge cross-section. These precast panels are attached to steel I-girders with a special cast-in-place shear pockets to form shear connections between them and the girders. Panels are joined together using transverse full-depth cast-in-place joints to maintain structural integrity. Figure 1.1 shows schematic diagram of such precast panels over two steel girders. While Fig. 1.2 shows a recent installation of such panels in Hwy 401/Mull Road Underpass using reinforcing steel bars in the panel as well as the joints.



(a) Plan

(b) Precast deck panel

(c) Bridge cross-section

Fig. 1.1 Prefabricated bridge system with full-width, full-depth, precast concrete deck panels

To make this system effective, two types of connections would be present, namely: shear connections between the steel girder and the precast deck panel and transverse joints between precast panels (Fig. 1.2). Composite action between deck panels and the girders is achieved via shear pockets. This system is attractive since it eliminates the overlays resulting in getting the bridge opened for traffic faster, especially on a deck replacement project. In this case, cast-in-place concrete is needed only at the joints between the prefabricated panels.

Rapid-set concrete mixes, which do not require skilled concrete placement and finishing workers, can be used for those joints. This system also eliminates field post-tensioning of the precast elements, thus shortening the construction schedule, lower the cost or the deck and simplifies the design with respect to avoid problems with long-term losses in pretensioning the slabs. Based on the literature review, the connection between the precast bridge deck panels and the steel girder, using the lumped shear studs and shear buckets, needs further investigation to stand on its load carrying capacity as opposed to that for the traditional cast-in-place slab connected to the steel girders using regularly spaced shear studs.

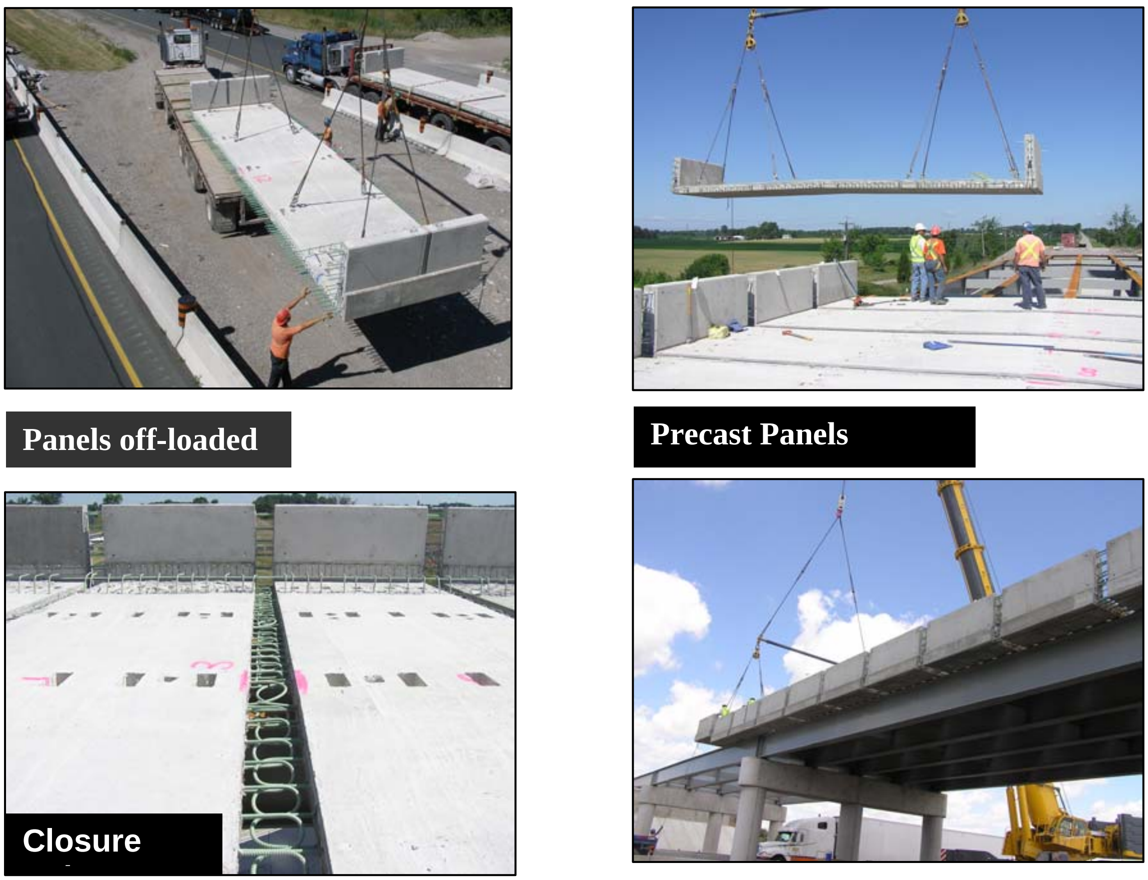


Fig. 1.2 Views of the Construction of Hwy 401/Mull Road Underpass Using Full-Depth, Full-Width, Precast Deck Panels with Reinforcing Steel (Source: MTO Bridge Office)

1.3. Research Objectives and Scope of Work

The objectives of this research were to conduct push-off tests on selected specimens of different shear stud configurations to determine their structural behaviour, crack pattern and ultimate load carrying capacities as compared to the traditional cast-in-place slab connected to the steel girders using regularly spaced shear studs.

The scope of work involved conducting brief literature review on the shear connector research and accelerated bridge construction. Eight reduced-scale specimens were erected and tested in the Structures laboratory. The results from the experiments were then used to discuss their structural behaviour and ultimate strength. Recommendations for the use of such connections in practice as well as extending this research were drawn at the end of this report.

1.4. Arrangement of the Report

Chapter II presents brief literature review of the previous work and the concept of shear connection between the concrete slab and the steel beams. Chapter III presents the layout and details of the eight specimens to be tested, instrumentations and test procedure. Chapter IV presents the experimental findings, while Chapter V presents the conclusions and recommendations for further research.

CHAPTER II

LITERATURE REVIEW

2 . 1 Introduction

This Chapter presents brief review of some literature related to the behaviour of the shear connectors between the concrete slab and the steel beams, followed by brief description of prefabricated bridge concept to accelerate bridge construction.

2.2 Shear Connector Behaviour

Shear Connector is a steel element used to resist horizontal shear between elements of a composite beam. To demonstrate the role and effect of shear connectors in a composite element, behaviour of the slab on girder (without shear connectors) and slab on girder (composite beam with shear connector) is demonstrated as follows. Figure 2.1 shows s simply-supported beam loaded with a uniformly distributed load along its span. The moment resistance of the system is the summation of the moment resistance of the girder and that of the concrete slab acting as two separate elements. As gravity loading is applied, deformation of the system commences, forming two independent tension compression zones in the two system elements. The figure shows the resulting stress and strain diagrams for both the concrete slab and the steel beam independently. Since the slab is not connected to the steel beam, the friction at the interface between the two materials will be overcome at a very low load increment, followed by an apparent slippage as shown in Fig. 2.1.

In case of concrete slab connected to steel beam using shear connectors as shown in Fig. 2.2, composite action will be in effect since the shear connectors will resist the shear force at the interface between concrete slab and the steel beam, thus no slippage will occur. In this case, the bottom portion of the composite section will be under tension, while the top portion is subjected to compressive stresses. In other words, shear connectors cause the stress level at the interface of the two materials to coincide. Providing shear connectors greatly improves the shear and moment resistance of the composite section.

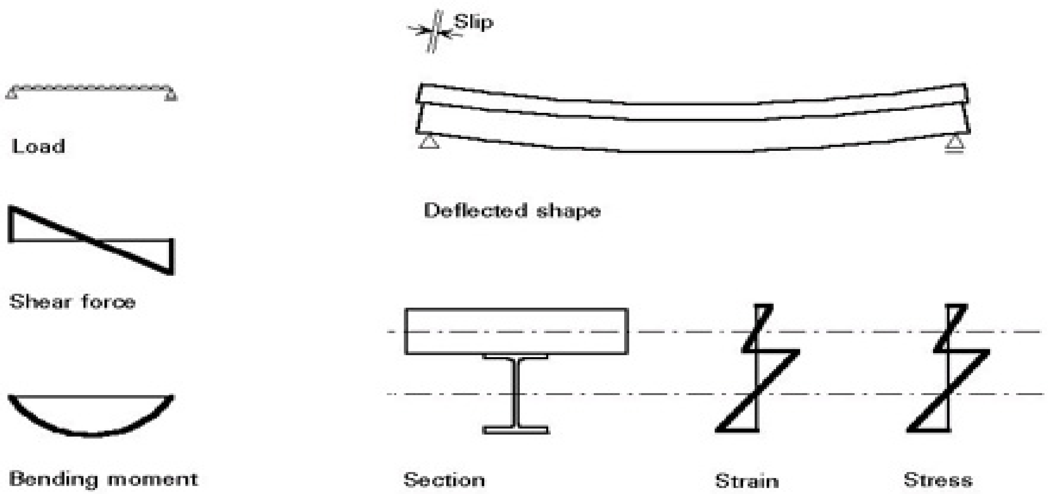


Fig. 2.1 Behaviour of non-composite concrete slab resting over steel beam
(http://www.corusconstruction.com)

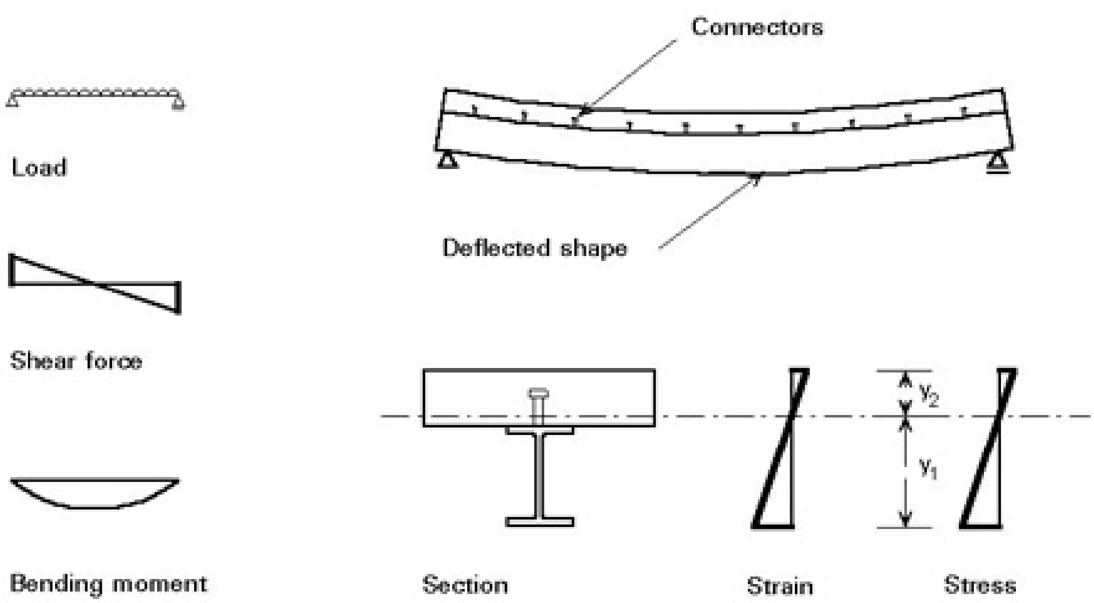


Fig. 2.2 Behaviour of non-composite concrete slab resting over steel beam
(http://www.corusconstruction.com)

Assuming that the fully-connected shear composite beam is performing under its elastic state, one can estimate the shear flow (the gradient of a shear stress force through the body) along the cross section of the element at any point along the length of it to be,

$$q = \frac{V_y Q_x}{I_x} \qquad (2.1)$$

Where:

q = the shear flow through a particular web section of the cross-section;

V_y = the shear force perpendicular to the neutral axis x through the entire cross-section;

Q_x = the first moment of area about the neutral axis x for a particular web section of the cross-section; and

I_x = the second moment of area about the neutral axis x for the entire cross-section.

From the above equation, for a simply-supported beam subjected to a uniform distributed loading along its length, a linear increase of shear (Solid line, Fig. 2.3) is expected as the beam behaves in its elastic state (q = 0.7 time the plastic failure load). Due to change in the bending moment and shear forces applied to the composite beam, rigid shear connectors are subjected to shear load relative to their location along the span of the beam. As the applied gravity load increases, approaching plastic failure load and the ultimate moment of resistance (q = 0.98 time the plastic failure load), shear load distribution pattern along the span of the beam differs (Dotted line, Fig. 2.3). This distribution pattern is due crushing of the concrete and steel sections along with the formation of plastic hinges in critical locations.(Kim, 2001). It should be noted that in practice, slippage between the concrete slab and steel sections increase with increase of the applied load. The flexibility of shear connectors and their ductile behaviour alters the shear distribution due to apparent effect of increasing slippage between concrete and steel at their contact surface. At the ultimate stage near failure, flexible shear connectors at the beam supports will be extensively deformed and yet are expected to carry significant shear load in the longitudinal direction.

Stiffness of the connection in relation to that of the steel and concrete sections (referred to as the interaction) must be taken into account. Beams with ultimately stiff connectors are considered to have full-interaction as oppose to more practical beams with flexible connectors having partial interaction. The location of the major shear force in a connector is usually at its end where it is welded to the steel beam. Relative movement of concrete slab to that of the steel beam is likely to cause an "S" shape deformation along the length of the flexible shear as shown in Fig. 2.4.

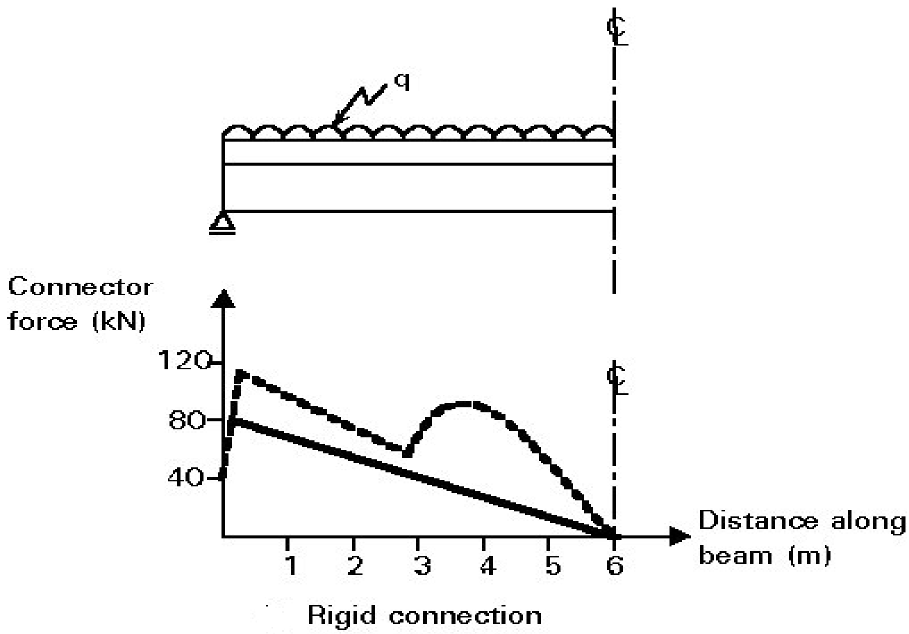


Fig. 2.3 Distribution of forces in the shear connectors at the elastic and failure stages of a composite beam

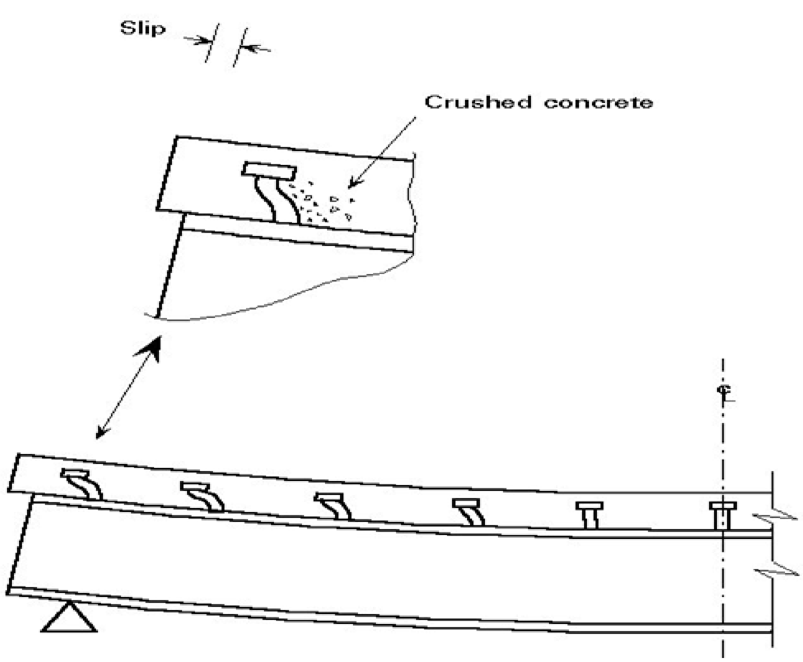


Fig. 2.4 View of the deformed shear connector and slippage at the concrete-steel interface

Ductility of the connectors along with the high level of stress experienced near the base of the connectors causes crushing of concrete as shown in Fig. 2.4 and explains the demonstrated behaviour of the section (slippage and deformation). Connectors with low slender ratio (short and stocky) have brittle behaviour under such loading thus considered undesirable. Another force to be resisted by shear connectors is direct tensile force. This force is induced by deferential bending stiffness of the concrete slab and that of the steel section, which accentuates separation (rising) tendency of the slab from the steel beam, espoused with the deformation of (shortened in height due to deformation) shear connectors. This induced force causes the accumulation of stress in proximity of the connector's head, which is depicted by the darker areas in Fig. 2.5. Concrete bearing failure is experienced at the connector's base proximity where as induced tensile force causes disintegration of concrete around the connector's head. Figure 2.6 showed schematic diagrams for the shear stud failure modes including the tensile failure (shank failure), embedment failure, slab cracking (cracking and crushing of concrete), or shear failure of the slab.

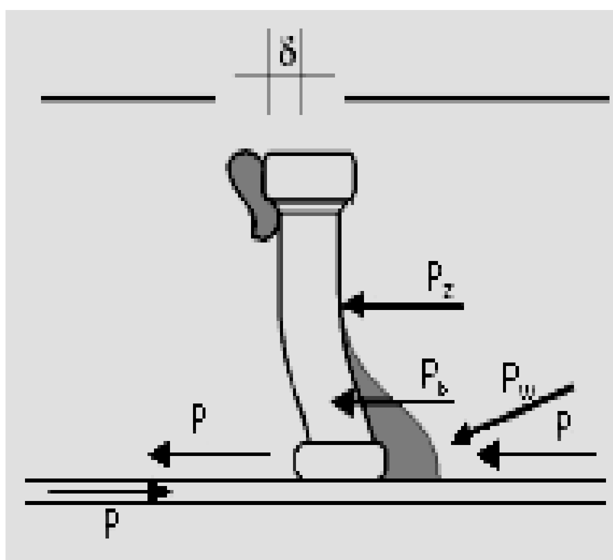
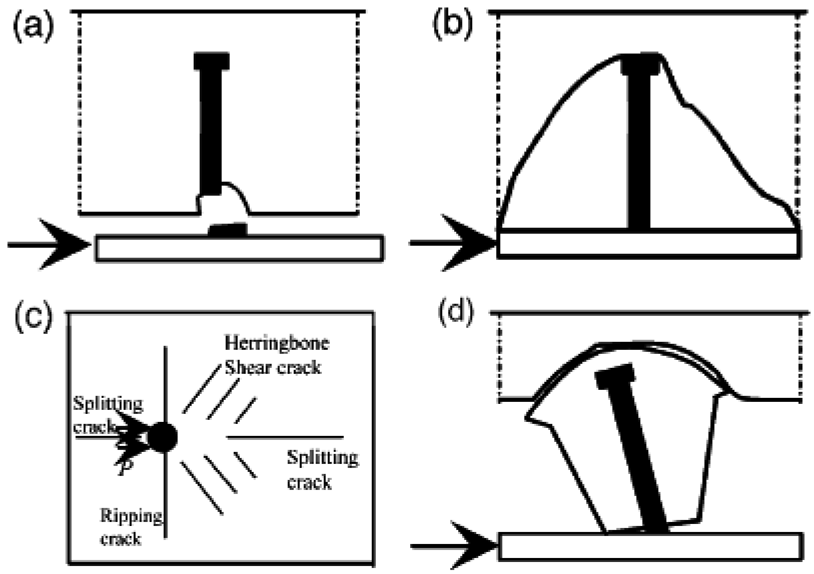


Fig. 2.5 View of areas of stress concentration and forces in a shear stud



Failure modes of stud shear connection. (a) Shank failure; (b) embedment failure; (c) slab cracking; and (d) shear failure of slab.

Fig. 2.6. Schematic diagrams of the failure modes of a shear stud

2.3. Experimental and finite-element studies on shear connector strength

To determine the shear stud capacity, pull-off tests can be conducted as shown in Fig. 2.7 (Kim, 2001). In this test, a steel beams is connected to two slabs, one from each side, using shear studs at a specified spacing. The steel beam is then subjected to applied load to collapse. The applied load is transferred to the concrete slabs through the shear connectors. The movement of the steel beam relative to the concrete slab is measured using dial gauges installed either on top of the concrete slab or on profile sheets close to the locations of the studs as shown in Fig. 2.8. This figure also shows the resulting load-slip relationship.

Other research involved the use of the finite-element modelling in investigating the structural behaviour of the shear connectors and correlate it with the experimental findings (Ellobodya, 2005). Figure 2.9 shows view of the finite-element modelling of a shear stud connector embedded in concrete. While Fig. 2.10 shows comparison between the experimental and finite-element modelling of the load-slip relationship of a stud connector (Kim, 2001). Figure 2.11 presents the concrete crack pattern and failure mechanism obtained from the finite-element modelling for multi-stud connection.

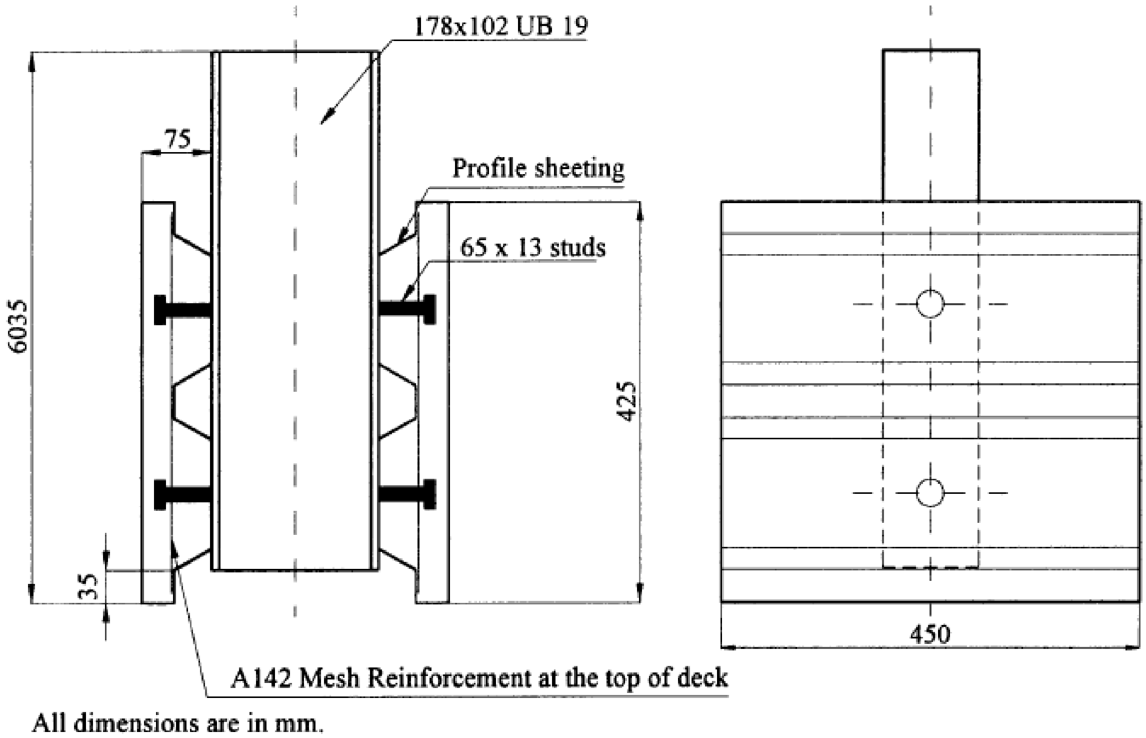


Fig. 2.7 Schematic Diagram of the push-off test specimen

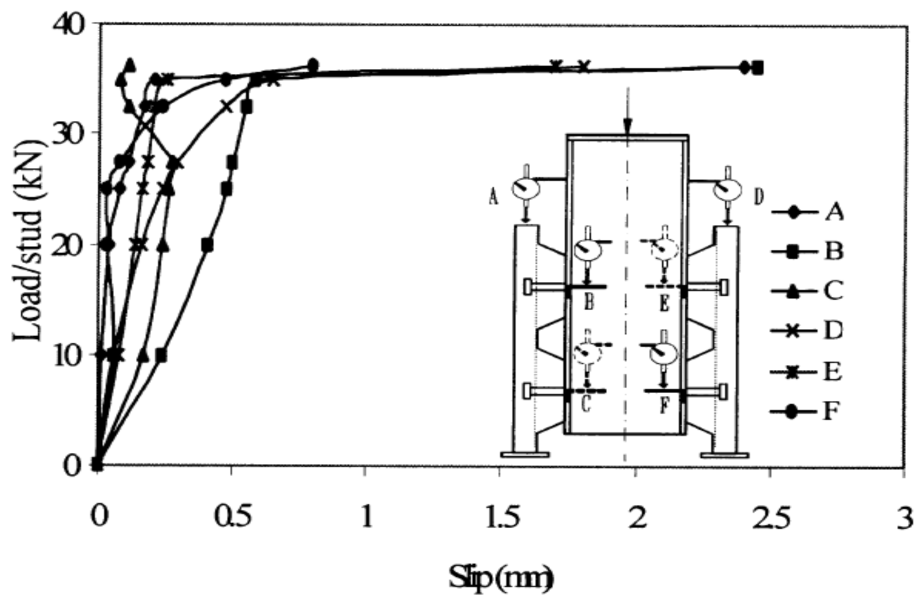


Fig. 2.8 View of the resulting load-slip relationship for push-off tests (Kim, 2001)

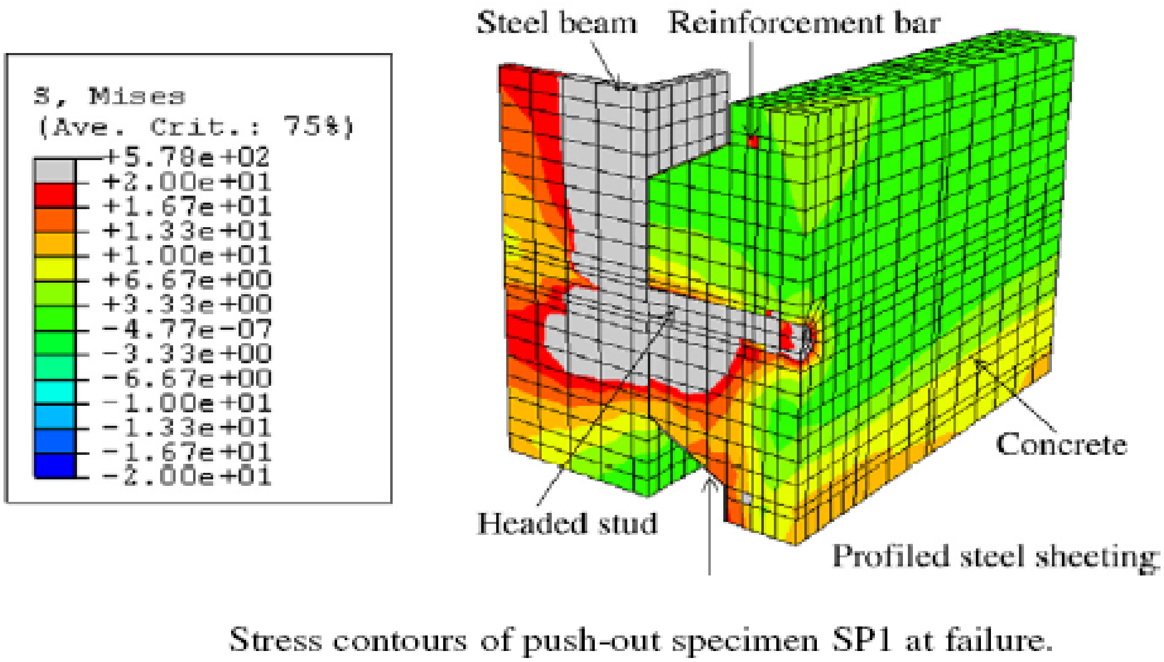


Fig. 2.9 View of the finite element modelling of stud connector showing stress contours (Ellobodya, 2005)

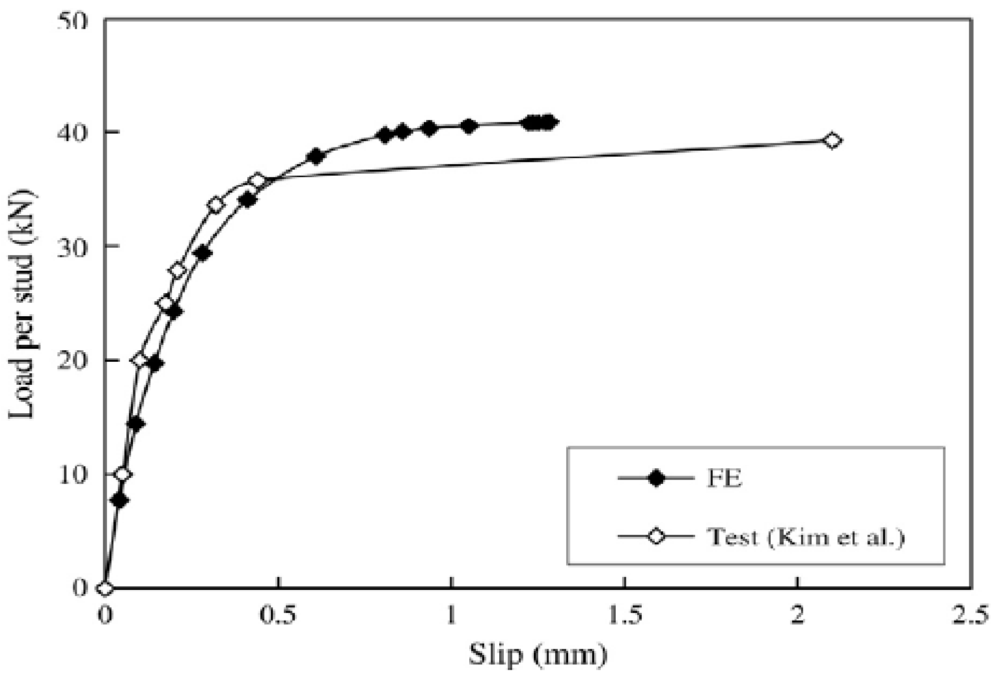
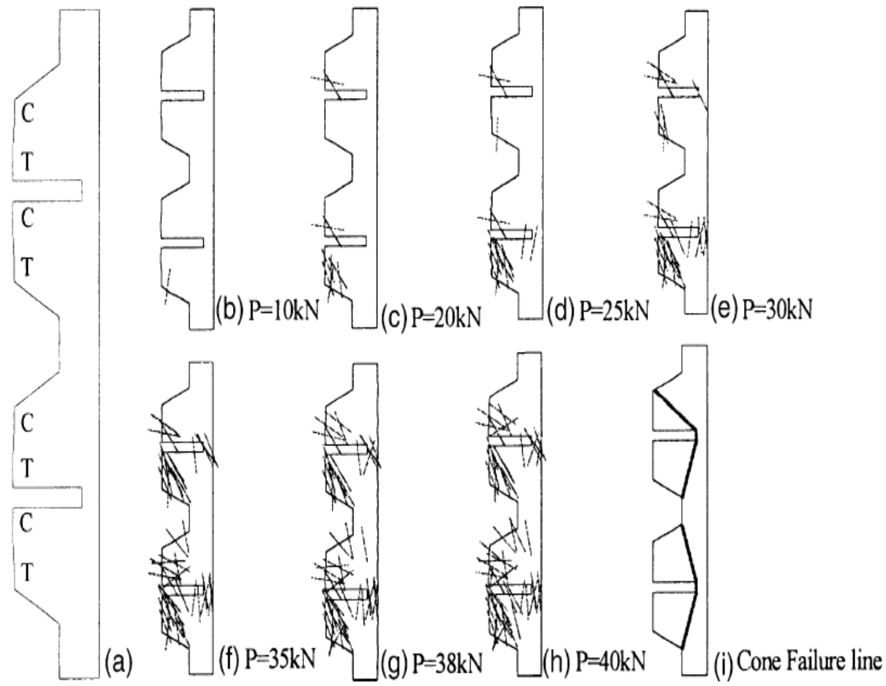


Fig. 2.10 Correlation between the FEA and experimental findings for load-slip relationship of stud connectors (Ellobodya, 2005)



Concrete crack pattern. C and T denote compression and tension respectively.

Fig. 2.11 Views of crack pattern and failure mechanism obtained from FEA modelling (Ellobodya, 2005)

2.4 Code Provisions for the Design of Shear Connectors

The Canadian Highway Bridge Design Code (2006) specified two equations to determine the factored resisting shear force carried by a stud connector. The first equation is based on crushing of concrete as follows:

$$q_r = 0.5\phi_{sc} A_{sc} \sqrt{f_{c'} E_c} \tag{2.2}$$

Where :

q_r = capacity of one shear stud(N/stud);

φ_{sc} = resistance factor of the stud

A_{sc} = cross-section area of a stud

f_c` = concrete compressive strength

E_c = concrete modulus of elasticity

The second equation for shear stud resistance is based on the tensile capacity of the stud itself (i.e. bending of the stud within the concrete section) as follows:

$$q_r = \phi_{sc} A_{sc} F_u \tag{2.3}$$

Where F_u = tensile strength of a stud and is usually taken as 415 MPa for commonly available studs.

The number of studs per row and number of stud rows, as shown in Fig. 2.12, can be determined using the above equations along with the available factored applied load in the stud group.

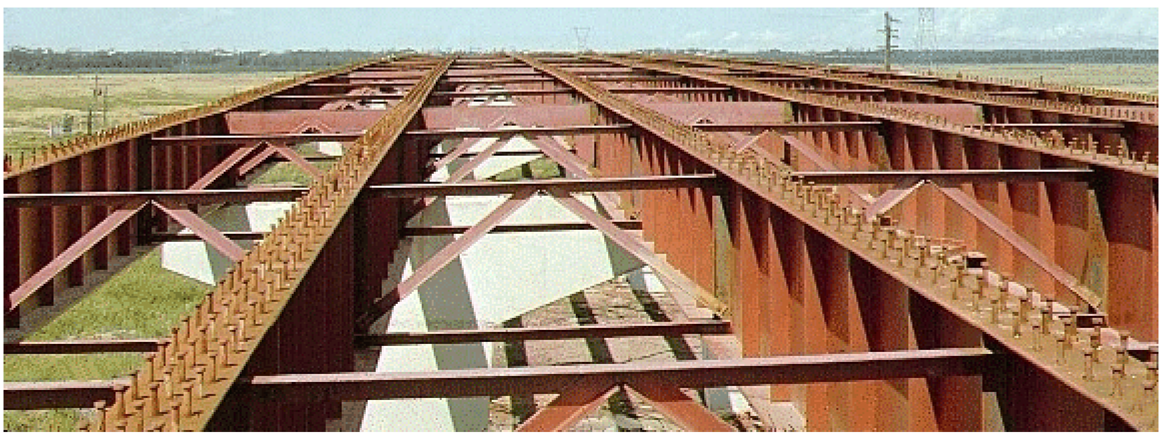


Fig. 2. 12. View of a steel girders in a bridge with shear studs

2.5 Brief History of Prefabricated Bridges

The involvement of the prefabrication industry in bridge construction consists primarily of providing some factory produced elements. Through mass production of the materials and reduction of on-site construction time, economic benefits are most often achieved. Prefabricated elements commonly produced are prestressed concrete piles, I-beams, box beams, channels, hollow and solid slabs, deck panels, steel I-beams (built-up members and rolled shapes), and box (trapezoidal) beams. Various forms of precast prestressed T-beams have evolved over the past few decades to build short-span bridges (Curtis, 1967; Kwei, 1967). These include contiguously placed single-T, double-T, and multiple-T sections and are suitable for bridges in the span range of 6 to 24 m. However, some single-T bridge sections can span up to 36 m. These precast sections are produced in standardized widths of 1.2, 1.8 and 2.4 m. The fully precast beams are transported to the site and erected adjacent to each other. V-joints between the edges of their flanges are filled with nonshrink mortor grout and are transversely post-tensioned to provide for lateral resistance and continuity for load transfer (Shahaway, 1990; Arokiasamy et al., 1991; Shahaway and Issa, 1992). Other mean of shear transfer can be achieved through the use of grouted keyways, transverse tie rods, or weld plates (Sprinkel, 1985). Some of these beams have projecting web reinforcement that is embedded in the cast-in-place slab to develop the composite action for live load. Others may be fabricated for use with or without concrete topping (Sprinkel, 1985).

For increased span capabilities, the Concrete Technology Corporation developed the bulb-T series, having a 1.2-m wide to flange and several standardized depths, in 1959. Arthur Anderson improved this design in 1969, developing the innovative decked bulb-T series with large standardized flange widths of 1.5 and 3 m, each with several standardized depths from 700 to 1900 mm, with span capabilities up to 57 m (Anderson, 1957, 1972, 1973). Placed contiguously, these girders provide a ready-make deck, eliminating the need for a closely cast-in-place deck. Anderson also developed the Washington series 14 bulb-T, which was standardized with some modifications in 1988 as the AASHTO-PCI bulb-T series (Geren and Tadros, 1994). Noteworthy are the details of the prefabricated, galvanized steel, K-shaped diaphragms, which are field-bolted to the steel plates anchored in these girders. Roller et al. (1995) have presented the results of testing a 21-m-long, 1.35-m-deep pretensioned high-strength concrete bulb-T girder having a 3-m wide top flange. The results showed that this girder has withstood more than 5 million cycles of fatigue loading and satisfied all serviceability requirements.

2.6 Prefabricated Bridge Deck Systems

Prefabricated decks offer advantages for deck construction because bridge components can be prefabricated off-site and assembled in place. Other advantages include excluding deck placement from the critical path of bridge construction schedules, cost savings, and increased quality as a result of controlled factory conditions. However, proper design and construction of the joints must be addressed to ensure adequate performance.

2.7 Partial Depth Bridge Deck Panels

Partial-depth prefabricated deck panels act as stay-in-place (SIP) forms and not only allow more controlled fabrication than fully cast-in-place decks, but also could increase the strength of the finished bridge owing to the use of prestressed panels. When panels are used, the bottom layer of the reinforcement in both the transverse and longitudinal directions that is present in a conventional cast-in-place, full depth, reinforced concrete bridge deck is eliminated. Few authors dealt with the composite action between girders and bridge deck with precast panels. Burns and Centennial (2001) carried out experiment on Type I AASHTO girders with composite deck. Two specimens used a full 200-mm thick cast-in-place, normal weight concrete deck and the other two specimens used 100 mm thick precast, lightweight concrete panels with 100 mm thick cast-in-place, normal weight concrete deck. The load-deflection curves and strengths for each pair of specimens are almost identical. Strain gauges placed across the width of the slab showed that the full width of the slab was effective in both cases. Based on the test results, full composite action, with or without the use of precast, prestressed concrete panels, can be assumed for both service load and strength calculations. Abendroth (1995), experimentally investigated the nominal flexure and shear strength of composite slab system with precast prestressed concrete panels as subdeck in bridge construction. Five full scale models of composite slab specimens were constructed and tested. Experimental results were compared with analytically results using yield line and punching shear theories and concluded that full-composite behavior was maintained between the reinforced concrete topping and the precast concrete panel and punching shear mode of failure governed the nominal strength of the slabs.

2.8 Full-depth Bridge Deck Panels

To rehabilitate the decks of heavily traveled bridges, full depth prestressed concrete panels are placed transversely on the supporting girders and post-tensioned longitudinally. Portions of a deteriorated deck can be removed during night operations and the full-depth panels installed in time to open the structure to morning traffic. Other deck systems offer similarly rapid construction methods with the advantages of reduced dead load and enhanced durability. Yamane et al. (1998) developed new full depth precast prestressed concrete bridge deck panel system. The newly developed system includes stemmed precast panels, transverse grouted joints, longitudinal post-tensioning and welded threaded and headless studs. A finite-element analysis was carried out to find out stresses in the deck panel and compare these stresses with experimental values. They constructed full scale prototype of the proposed precast panel system and tested under fatigue and ultimate loading. They concluded that the performance of the proposed system meets all the structural requirements for bridge decks and comparable to exodermic bridge deck system in weight and much less expensive. An exodermic bridge deck consists of a fabricated steel grid for the bottom portion and a reinforced concrete slab for the top portion. A part of the steel grid portion extends upward into the reinforced concrete in order to achieve a composite deck. Punching shear, rather than flexure, was the mode of failure for the proposed system.

In 1999, 1301.12 m^2 of deteriorating bridge deck of Route 7 over Route 50 bridges in Fairfax County, Virginia, USA, required replacement [McKeel (2002)]. Virginia's DOT opted to use full-depth prefabricated concrete deck panels to satisfy community concerns with respect to reduction in the level of service. Operating only at night, work crews saw cut sections of the existing deck, lifted and removed them by crane, and immediately installed new deck panels that matched the deck cavity. A rapid-setting concrete overlay was then placed, and after only 3 hours the bridge was able to support full traffic. The bridge was completely open to traffic during the day. In 2001, Route 29 over Sugar Creek in Illinois required the redecking of an existing 77.13 m-long, 11.4 m-wide five-span bridge (McKeel, 2002). The existing steel beams were reused and made composite with the prefabricated deck panels. A total of 29 panels were laid across the length of the bridge. The panels were connected by shear keys and post-tensioned longitudinally. Traffic delays were minimized as a result of the speeding up of the construction time. Tadros et al. (2002) studied the behavior of debonded shear key system for girder and full depth deck connections. The system has the advantage of facilitating future deck removal, while protecting the top flange of the girder from damage, which is particularly significant for bridges in cold climates where deck concrete is subjected to deterioration due to freeze-thaw cycles and deicing chemicals.

2.9 Prefabricated Girder Connections

The PCI committee on connection details (1995, 1998) published typical details and its design method for standard connections for precast and prestressed concrete Double-Tee girders. The connection included longitudinal and transverse joints. The criteria for the connection designs are strength, ductility, volume change accommodation, durability, fire resistance, constructability, aesthetics, and seismic requirement. The flange–to-flange double-T beam connection is made of an inclined steel plate anchored to the concrete flange using a special shape steel rod. This connection transfers the shear through welding of plates with a rod. Also, same details were presented in Applied Technology Council (1981) report ATC-8 for Tee beam flange connection with some modification. The Applied technology council report ATC-8 (1981) discussed some connection details for prefabricated concrete building element connections to resist earthquake loading. In one of these details, rebar hooks extend from each panel and are connected using longitudinal rebars and concreted with cast-in-place concrete. Other detail shows an intermittent connection for floor panel, consisting of a steel plate embedded in the panel at intervals and welded together with a connecting rod. The report presented a limited slip bolted connection for concrete floor panels. This connection consists of box embedded in the concrete panels at intermediate location and connected together with slotted plate and bolts, which permit some allowance for the slip. The report also presented embedded chord reinforcing details which contains embedded angle in double-tee beam flange welded with chord reinforcements and the tee flanges are then joined by a steel plate welded to angles.

Pincheira et al. (1998) carried out pilot series of tests on double-tee flange-to-flange connector to examine the strength and deformation capacity of connectors subjected to multi-axial and cyclic loading. The connector consisted of a steel plate with two filet-welded reinforcing bars embedded in 50 mm thick concrete slab, which was very similar to the PCI [PCI committee on connection details (1995, 1998)] standard details and the one presented in ATC-8 report (Applied technology Council, 1981). They observed moderate to high levels of deformation ductility under monotonic loading, while under cyclic loading, the deformation capacity and ductility of the connector were limited.

Arockiasany et al. (1991) studied fatigue strength of joints in a precast prestressed concrete double-tee bridge. 1:3.5 scale model of a two span, transversely and longitudinally post-tensioned, continuous double-tee beam system was tested in static and fatigue loading. Constant amplitude fatigue loading was applied on the model at typical locations simulating HS20-44 AASHTO (1998) truck loading. Structural integrity of the bridge system was checked and experimental deflection of the system was compared with the finite element analysis results. The ultimate load, computed from plastic analysis, was found to be in good agreement with the measured value. Researchers concluded that double-tee bridge system, assembled with post-tensioning in both the longitudinal and transverse directions, showed monolithic behavior under both static and fatigue loading. Bridge system was maintaining its structural integrity after 8 million cycles.

Hariatmadar (1997) studied seismic response of connection in precast concrete double-tees. He constructed thirty five specimens of five types of connections and tested under monotonically increasing shear force, reversed cyclic shearing and axial forces and various combinations of reversed cyclic shear and axial forces until failure using displacement control. Connections consist of angle welded with either anchor bars or headed studs or combination of both. He also developed design interaction curves and associated equation for each connection type and developed practice-oriented method to determine the connection strength under shear and axial forces and combination of these forces at joint between elements in precast system.

Hofheins et al. (2002) studied Behavior of welded plate connections in precast concrete panels under simulated seismic loads. Ten precast concrete wall panel assemblies were tested under in-plane lateral cyclic loading for loose-plate connection located in the vertical joint between panels. Shear loads were transmitted through the embedded plate to the surrounding concrete by three mechanisms, namely: (i) friction between the embedded plate and concrete; (ii) bearing of the end of the embedded plate on the concrete; and (iii) interaction between studs and concrete. Tests were performed by applying a quasi-static cyclic load to three precast hollow-core wall panels connected together with two loose-plate connectors at each vertical joint. Each loading step consisted of three cyclic load increments to simulate the effect of an earthquake. They concluded that loose plate connection can resist relatively high shear forces, the connection fails in a brittle manner when the deformed anchor bars tear free from the embedded angles, the connection is not suitable for the high seismic region, if it should be designed to remain elastic, and connection should modified to make it ductile by providing more surface area for concrete bearing.

Bakht et al. (2001b) carried out two field tests on shear connected concrete plank bridges using welded shear key. After testing these bridges, they developed a reliable and rigorous method of analysis of shear in the welded shear key. Based on this method, they developed simplified graphical method to rapidly predict the maximum transverse shear forces in the shear keys of any of the bridges under consideration. Bakht and Mufti (2001a) presented load distribution in shear-connected concrete plank bridges. The main purpose of the testing was to determine the suitability of various methods of bridge analysis. They carried out two field tests and concluded that the articulated plate method is suitable for analyzing the bridges under consideration, but only after the longitudinal torsion rigidity of the planks is reduced suitably to account for the lack of torsional resistant at their ends.

CHAPTER III
EXPERIMENTAL STUDY

3.1 Introduction

This chapter describes the experimental program conducted in this study. It includes the geometries of the test specimens, material properties, instrumentation and test procedure. The objective of this test program is to experimentally examine different scenario of the connection between the full-depth precast concrete bridge deck panel and the steel girders using clustered shear studs as compared to the traditional cast-in-place slab connected to the steel girder using regularly spaced shear studs.

3.2 Description of the test specimens

Eight specimens were erected to represent one-third scale of a bridge prototype. Table 3.1 shows summary of the specimens considered in this study. To facilitate the application of the load on the specimens, two identical concrete slabs were connected to a steel beam, one to each flange. Each slab was 600 mm wide, 800 mm deep and 75 mm thick and oriented vertically in the direction of load application. These slabs were connected to a W150x22 steel beam representing the top flange of the steel girder in practice. Each slab was reinforced with 5- 6 mm diameter steel bars each direction on the exterior face and 6- 6 mm diameter steel bars each direction on the interior face. Figure 3.1 shows schematic diagram of such configuration.

As shown in Table 3.1, two specimens, PO-1A and PO-1B, were erected to represent the bridge built with cast-in-place deck slab connected to the steel girder using regularly-spaced studs. In this case, 12 mm diameter shear studs were installed at 200 mm longitudinal spacing in two rows. Fig. 3.2 shows an elevation of these test specimens along with stud connector arrangement.

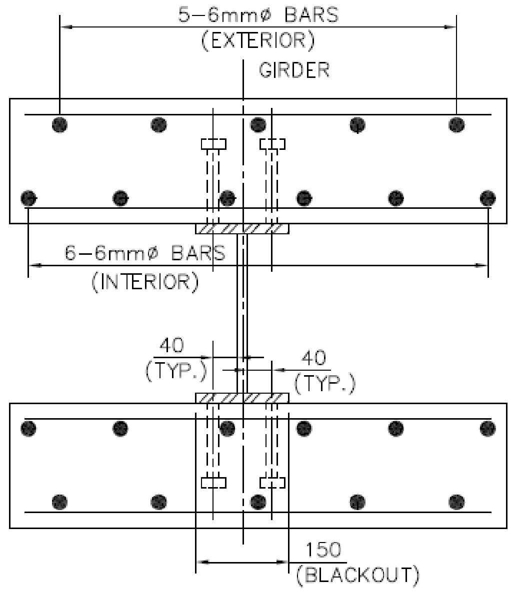


Fig. 3.1 Schematic diagram of the test specimen showing slab reinforcement and stud locations

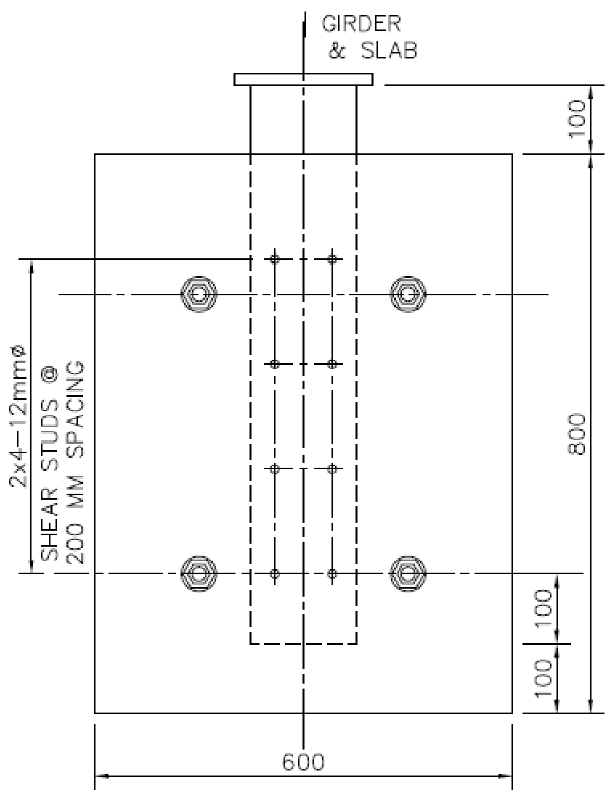


Fig. 3.2 Elevation of specimens PO-1A and PO-1B showing arrangement of regularly spaced studs

On other specimens, shear studs were clustered, allowing significantly larger spacing between each cluster. In practice shear pockets are to be prepared in the full-depth precast deck slab panels at the locations of the stud connector clusters. After laying down the precast panels over girders, shear pockets are filled with proper grouting materials. Figure 3.3 shows elevation of the last six specimens listed in Table 3.1. In this case, 8 studs were clustered in four rows with 2 studs per row. Stud diameter was taken 12 mm, with spacing of 50 mm within the stud pocket. It should be noted that this stud cluster was installed at the mid-height of the steel beam as shown in Fig. 3.3.

Specimens PO-2A and PO-2B are identical and represent the case of shear stud cluster embedded in cast-in-place (CIP) slab. However, specimens PO-3A and PO-3B represent the case of shear stud cluster with precast concrete deck slab on which cast-in-place concrete was cast in the shear pocket. It should be noted that the slab steel rebar were continuous from one side of the slab to the other side even through the shear pocket. Specimens PO-4A and PO-4B were identical to specimens PO-3A and PO-3B except that the steel rebar in the concrete slab did not extend through the shear pocket. This forth set of specimens is meant to examine the pop-out strength of the grouting material in the shear pocket without reinforcement to keep it in location, given the common practice of having the shear pocket conical or pyramidal in shape in practice. To facilitate slippage between the steel beam and the concrete slabs, a recess of a 100 mm between the slab and the lower end of steel girder was introduced. In addition, the steel beam was projecting 100 mm over the top surface of the concrete slab to allow for load application.

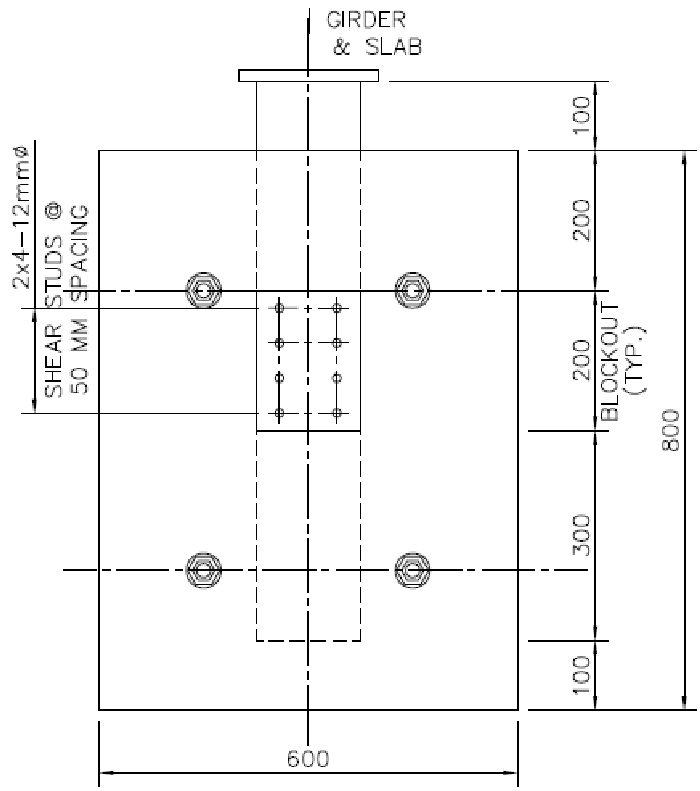
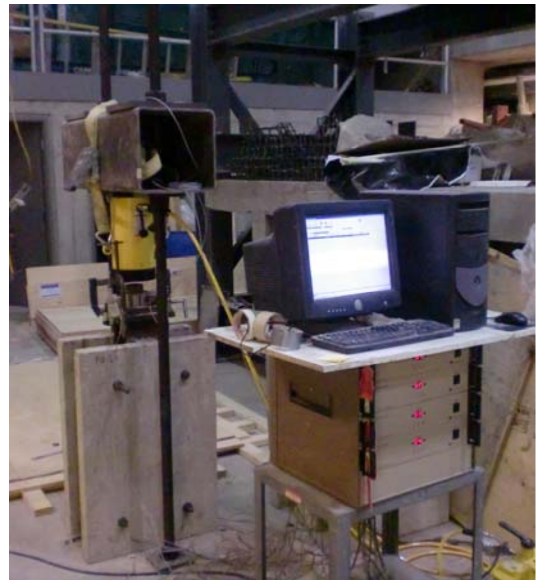


Fig. 3.3 Elevation of specimens PO-2A and PO-4B showing arrangement of clustered studs

3.3 Push-off Test Setup, Instrumentation and test procedure

A self-contained frame available at the structures laboratory of Ryerson University was utilized to conduct the push-off tests. Figure 3.4 shows view of the test setup where a jacking load was used to apply the load on the top of the steel beam. A 400,000 Ib load cell was used to measure the applied loading. To measure possible slip between the concrete slab and the steel beam, two LVDTs were installed at the top of the concrete slabs as shown in Figs. 3.5 to 3.7. Other two LVDTs were installed near the lower end of the steel beam to record its movement towards the laboratory floor. Four rods were bolted (snug tight) on all four corners of the samples connecting the two slabs, thus minimizing eccentric load application and stress distribution within the slabs. Each specimen was gradually loaded in increments up to 100 kN, then the load was released. Then, the load was applied again till it reached 200 kN. This step is repeated by increasing the reached load in one step by 100 kN in the following loading steps till failure occur.

(a)



(b)

(c)

Fig. 3.4 Views of the push-off test setup

Fig. 3.5 View of LVDT locations in the push-off test

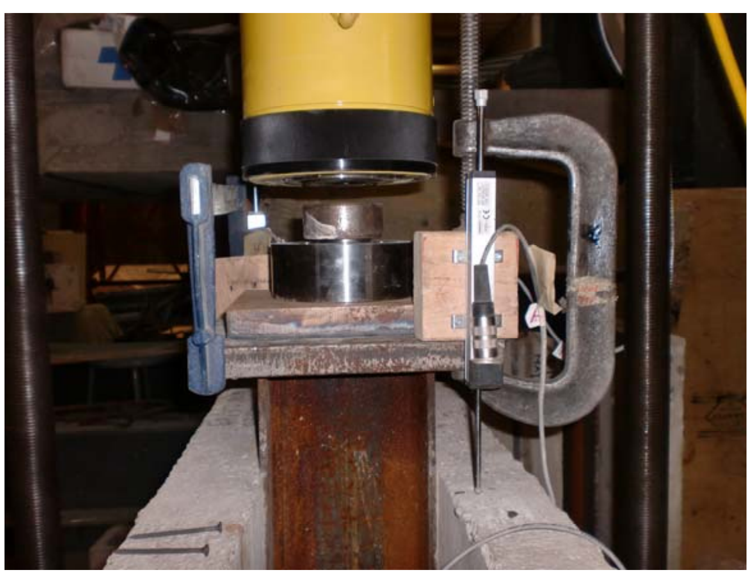


Fig. 3.6 View of LVDT on top of the concrete slab

Fig. 3.7 View of LVDT near bottom of steel beam

Table 3.1 Description of Test Specimens

Specimen	Description
PO-1A	Regularly-spaced shear studs with cast-in-place (CIP) concrete slab as control specimen
PO-1B	Regularly-spaced shear studs with cast-in-place concrete slab as control specimen
PO-2A	Shear stud clusters with CIP concrete slab
PO-2B	Shear stud clusters with CIP concrete slab
PO-3A	Shear stud clusters with precast concrete slab and CIP concrete in the shear pocket with steel rebar in the slab extending through the shear pocket
PO-3B	Shear stud clusters with precast concrete slab and CIP concrete in the shear pocket with steel rebar in the slab extending through the shear pocket
PO-4A	Shear stud clusters with precast concrete slab and CIP concrete in the shear pocket with steel rebar in the slab but not extending through the shear pocket
PO-4B	Shear stud clusters with precast concrete slab and CIP concrete in the shear pocket with steel rebar in the slab but not extending through the shear pocket

CHAPTER IV
EXPERIMENTAL RESULTS

4.1 Introduction

Eight reduced-scale specimens simulating the full-depth precast concrete bridge deck panel to steel girder connection were erected and then tested to-collapse to investigate their structural behaviour, crack pattern and ultimate load carrying capacity. The first two specimens were identical and represent the case of cast-in-place concrete slab connected to the steel girder using regularly spaced shear studs, while the other simulated different scenario of the connection between the full-depth precast concrete bridge deck panel and the steel girders using clustered shear studs. The experimental findings in this chapter are presented in the form of the crack pattern at failure, the load-slip relationship and the ultimate load carrying capacity.

4.2 Specimens with CIP slab and Regularly Spaced Shear Studs

Specimens PO-1A and PO-1B were identical and represent the case of cast-in-place slab connected to the steel beam using regularly spaced shear studs. Figure 4.1 shows views of the crack pattern at failure for specimen PO-1A. It can be observed that cracks were developed almost parallel to the load direction. This pattern resulted from splitting forces in concrete at the stud connector-concrete interface. It was observed that first crack appeared at 300 kN, while the ultimate load was recorded at 900 kN. With respect to load-slip relationship, data acquisition system channels 18 and 19 recorded the vertical displacement of the concrete slabs, while channels channel 16 and 17 recorded the vertical displacement of the steel beam from its front and back sides, respectively. Figure 4.2 depicts the applied load-slip relationship for specimen PO-1A. It can be observed that considerably similar behaviour was recorded from all LVDTs. The load-slip relationship was observed to have an straight line behaviour till a load of about 700 kN followed by a nonlinear relationship till failure. The maximum slip at recorded at failure to be approximately 1.45 mm.

(a) Front side (b) Back side

Fig. 4.1 Views of the crack pattern at failure for (a) the front side and (b) the back side of specimen PO-1A

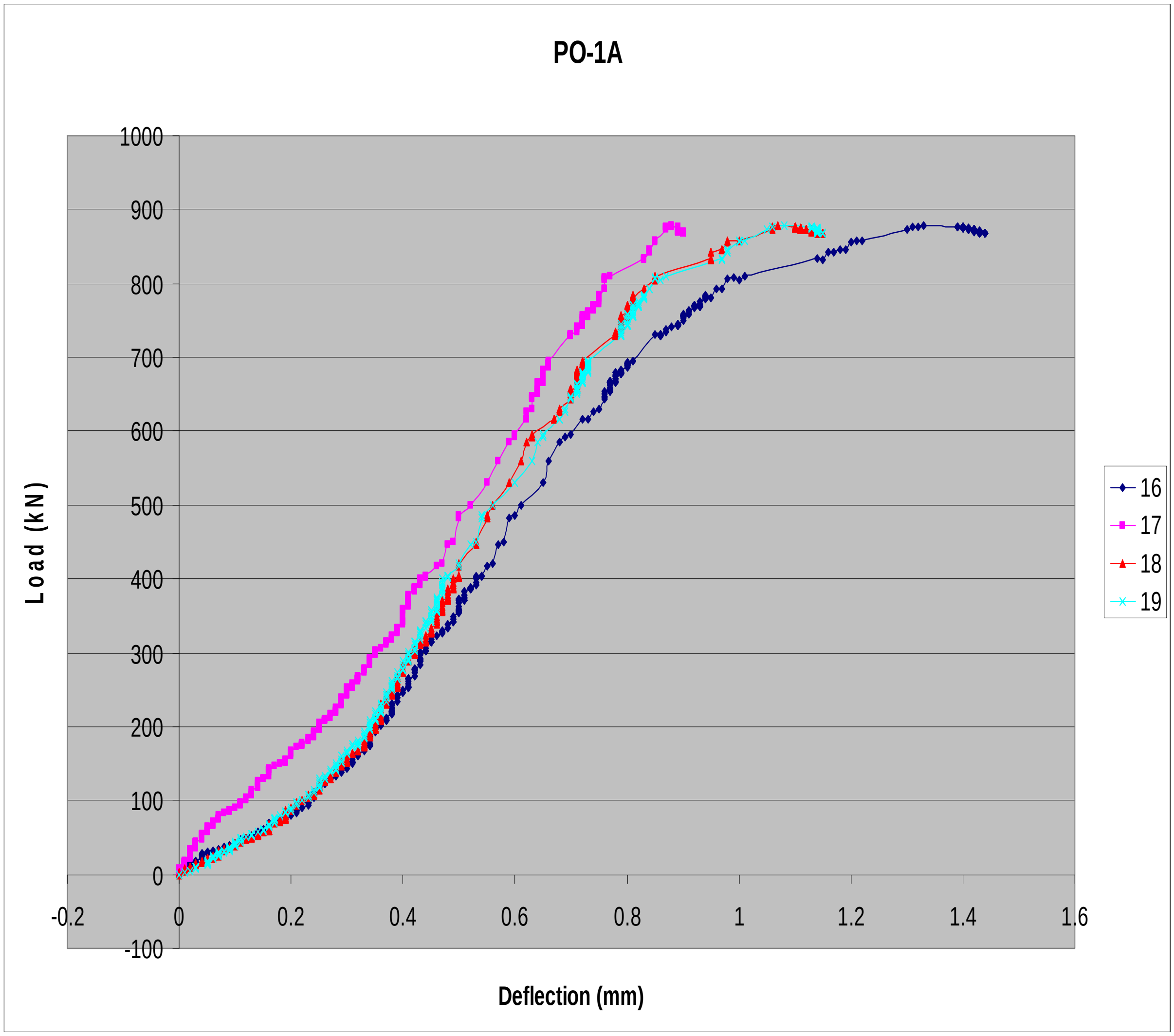
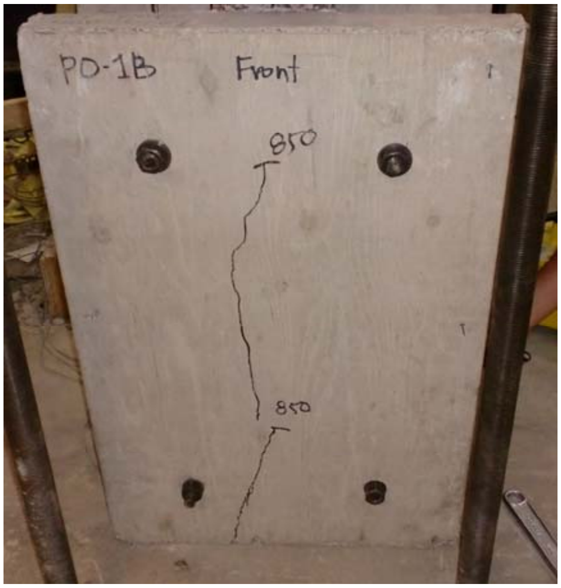


Fig. 4.1 Load-slip relationship for specimen PO-1A

Specimens PO-1B showed similar behaviour with respect to cracks due to splitting forced in concrete. However, these cracks appeared only at 850 kN load and on one side of the specimen only. Figure 4.3 shows such crack pattern for specimen PO-1B. The ultimate load was recorded as 880 kN. Figure 4.4. depicts the applied load-slip relationship for specimen PO-1B, which appear to be identical in shape to that for specimen PO-1A except that the maximum recorded slip was approximately 2.2 mm.



(a) Front side (b) Back side

Fig. 4.3 Views of the crack pattern at failure for (a) the front side and (b) the back side of specimen PO-1B

4.3 Specimens with shear stud cluster embedded in CIP slab

Specimens PO-2A and PO-2B were identical and represent the case of shear stud clusters embedded in cast-in-place slab. Figures 4.5 and 4.6 shows views of the crack pattern at failure for specimen PO-2A. It can be observed that cracks were initiated at a load of 200 kN and failure occurred at 860 kN load. Some of these crack all almost parallel to the direction of the force as a result of splitting forces in concrete. However, local crack was recorded normal to the direction of load at 200 kN load indicating excessive bearing strength on concrete at the back side of the stud cluster. Also, Figure 4.7 shows a sign of slippage at the interface between the slab and the steel flange in the direction of the load. Figure 4.8 depicts the load-slip relationship for specimen PO-2A that is similar in nature to those for specimens PO-1A and PO-1B. However, the maximum slip reached at failure was approximately 5.2 mm, with an average value of approximately 2.5 mm, given the large change in LVDT readings.

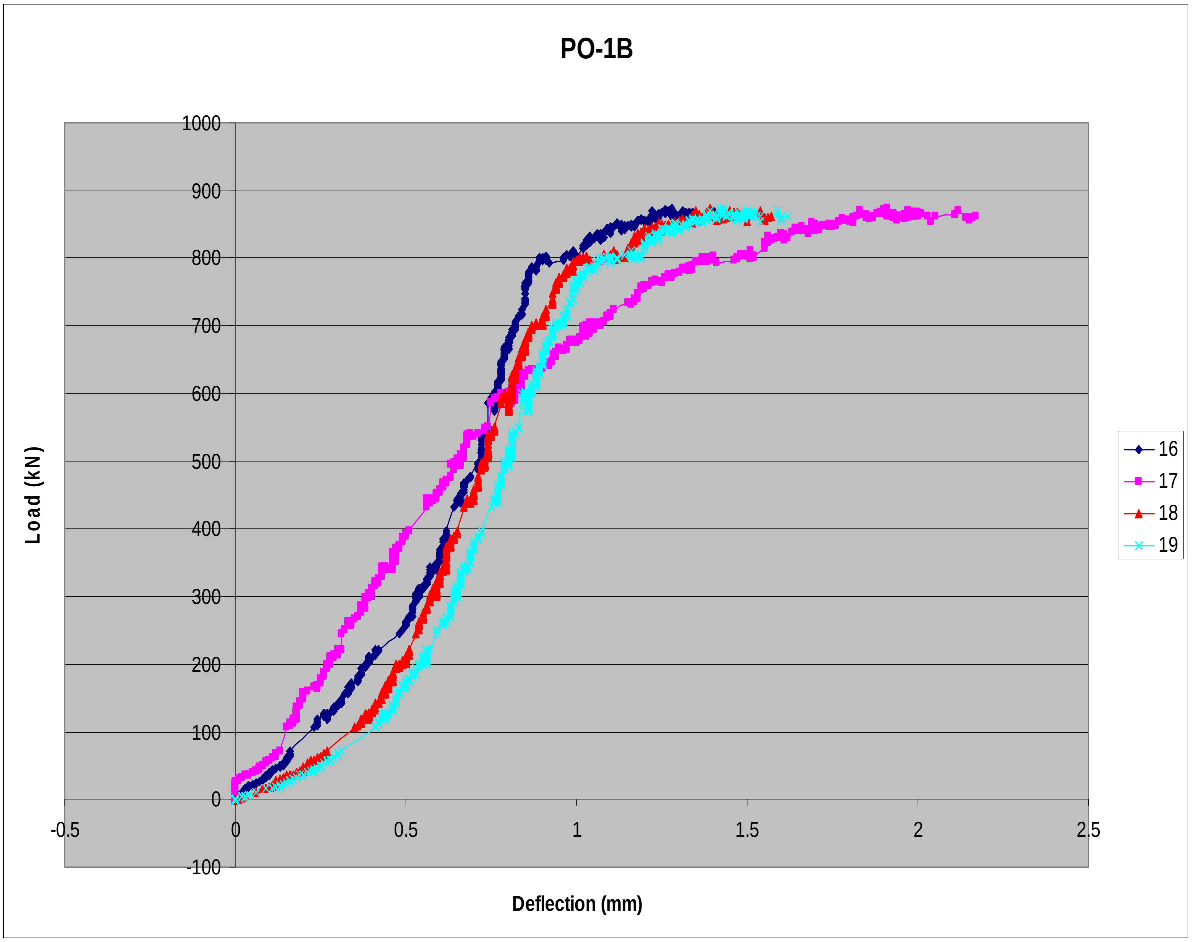


Fig. 4.4 Load-slip relationship for specimen PO-1B

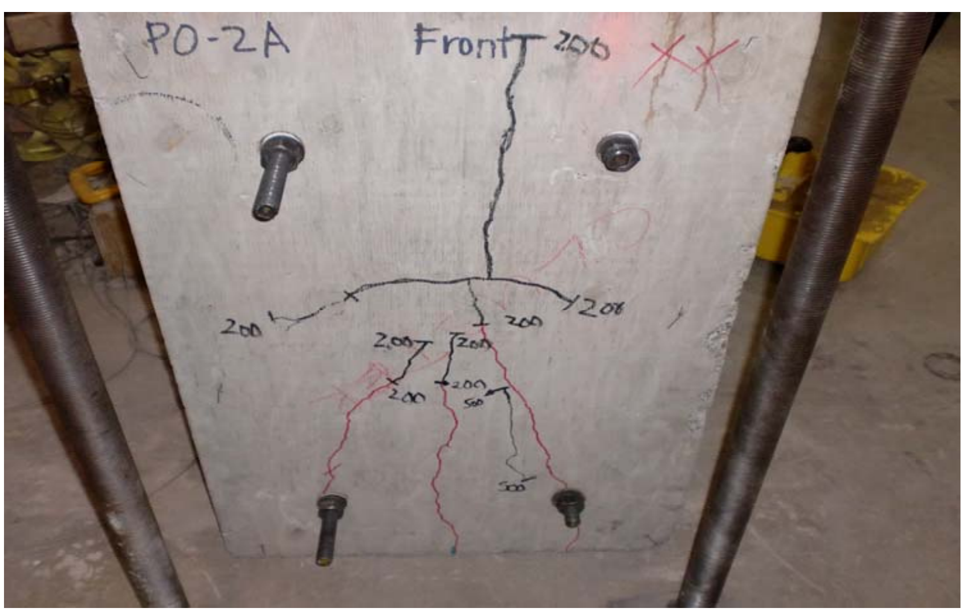


Fig. 4.5 View of the crack pattern at failure on the front side of specimen PO-2A

Fig. 4.6 View of the crack pattern at failure on the back side of specimen PO-2A


Fig. 4.7 View of the splitting crack at failure along the interface between the concrete slab and the steel flange of specimen PO-2A

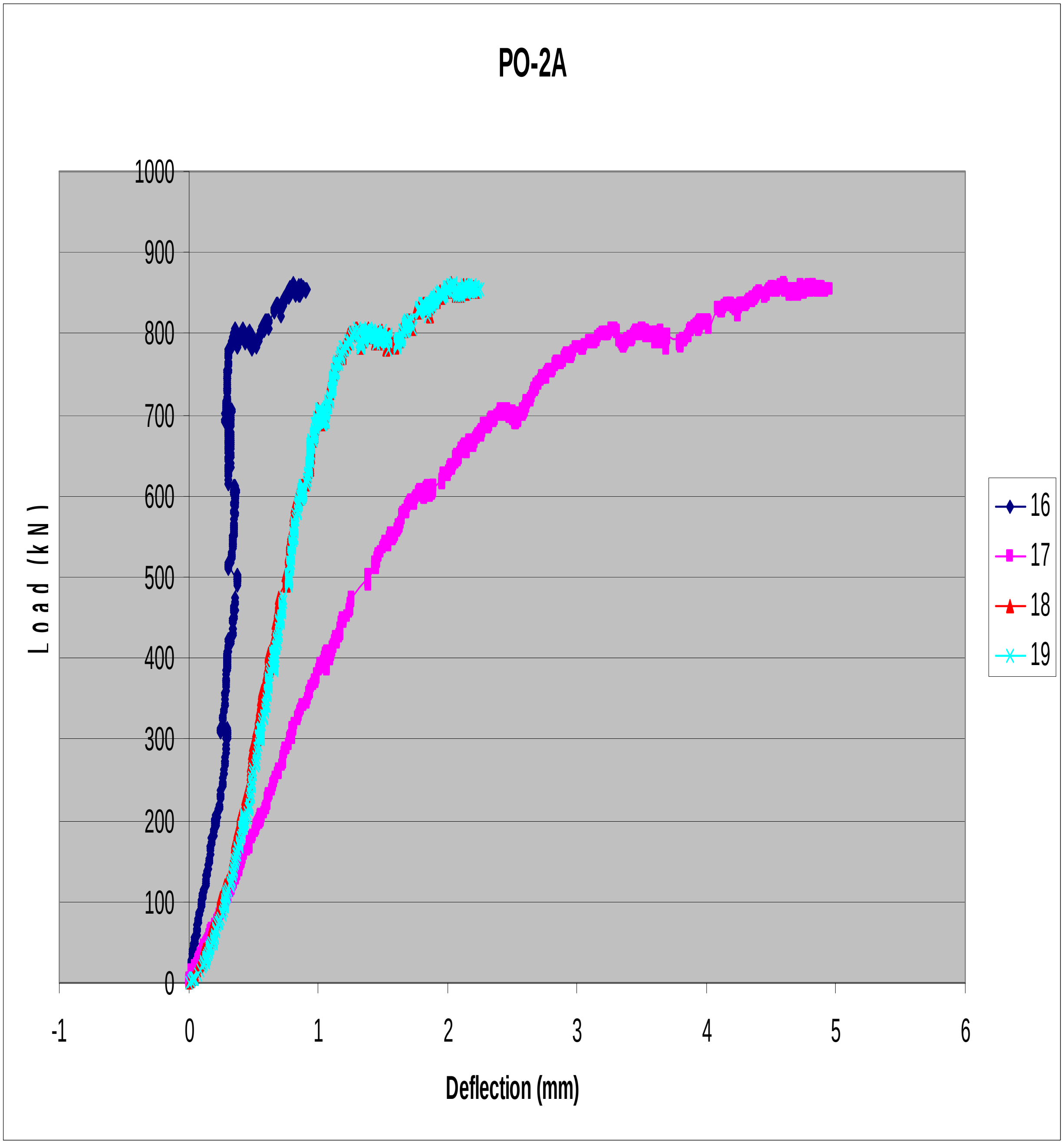


Fig. 4.8 Load-slip relationship for specimen PO-2A

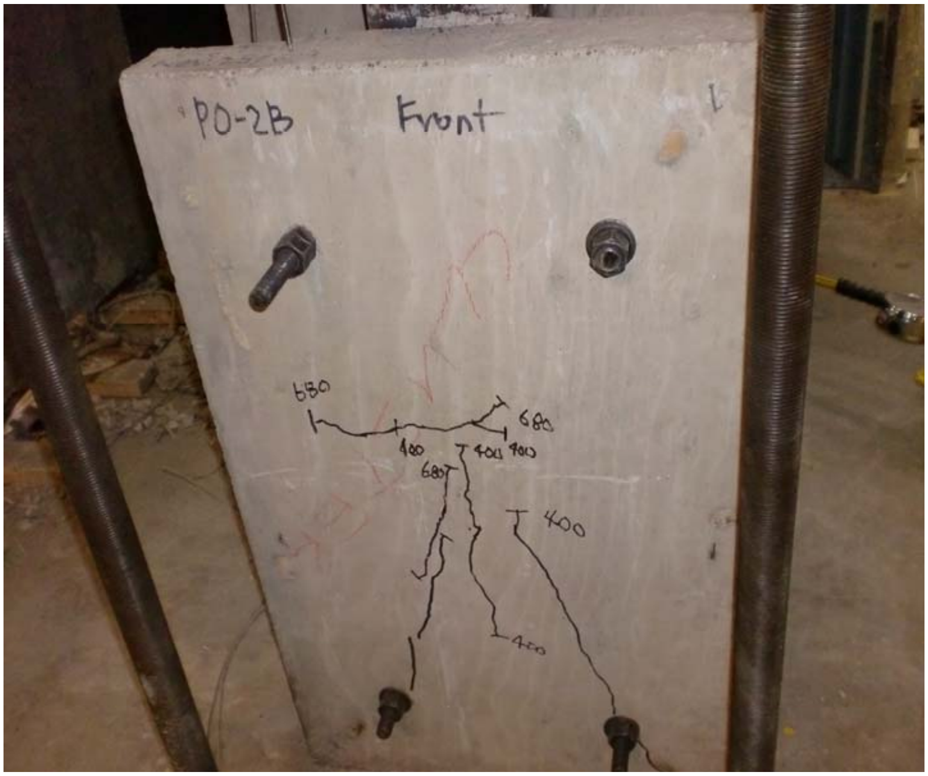


Fig. 4.9 View of the crack pattern at failure on the front side of specimen PO-2B

Fig. 4.10 View of the crack pattern at failure on the back side of specimen PO-2B

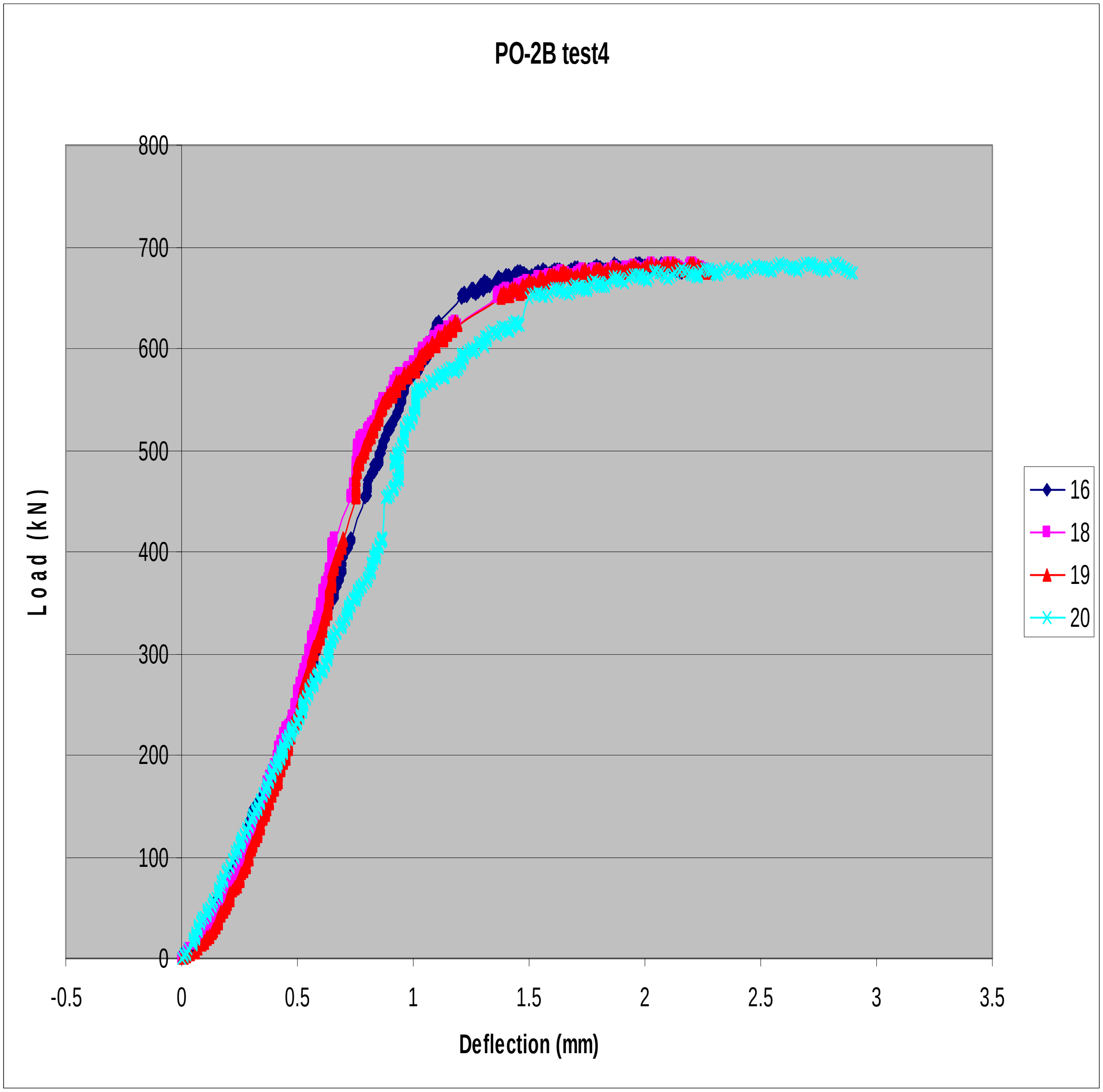


Fig. 4.11 Load-slip relationship for specimen PO-2B

In case of specimen PO-2B, Figs. 4.9 and 4.10 shows views of the crack pattern at failure on the front and back side of the specimen, respectively. It can be observed that cracks were initiated at a load of 400 kN and failure occurred at 680 kN load. Some of these crack all almost parallel to the direction of the force as a result of splitting forces in concrete. However, local crack was recorded normal to the direction of load at 680 kN load indicating excessive bearing strength on concrete at the back side of the stud cluster at failure. Figure 4.11 depicts the load-slip relationship for specimen PO-2B that is similar in nature to those for specimens

PO-1A and PO-1B. However, the maximum slip reached at failure was approximately 3.0 mm.

4.4 Specimens with shear stud cluster with precast concrete slab and CIP concrete in the shear pocket

Specimens PO-3A and PO-3B were identical and represent the case of shear stud clusters welded to the steel beam with a precast concrete panel resting over the steel beam and a shear pocket filled with cast-in-place concrete. Figures 4.12 and 4.13 shows views of the crack pattern at failure for specimen PO-3A. It can be observed that cracks were initiated at a load of 200 kN at the edges of the shear pocket propagating away from its ends. The failure load was recorded at 810 kN. Some of recorded cracks at 500 kN were almost parallel to the direction of the force as a result of splitting forces in concrete. However, local crack was recorded normal to the direction of load at 600 kN load indicating excessive bearing strength on concrete at the back side of the stud cluster. It should be noted that some of the horizontal and vertical cracks occurred in the precast slab propagated into the cast-in-place concrete in the shear pocket as depicted in Fig. 4.12. Figure 4.14 depicts the load-slip relationship for specimen PO-3A that is similar in nature to those for specimens PO-1A and PO-1B. However, the maximum slip reached at failure was approximately 4.0 mm, with an average value of approximately 2.0 mm, given the large change in LVDT readings. Based on LVDT readings recorded in Fig. 14 and crack pattern recorded in Figs. 4.11 and 4.12, one may observe that there was some sort of eccentricity on the applied load or specimen configuration that led to such unsymmetrical results.

Fig. 4.12 View of the crack pattern at failure on the front side of specimen PO-3A

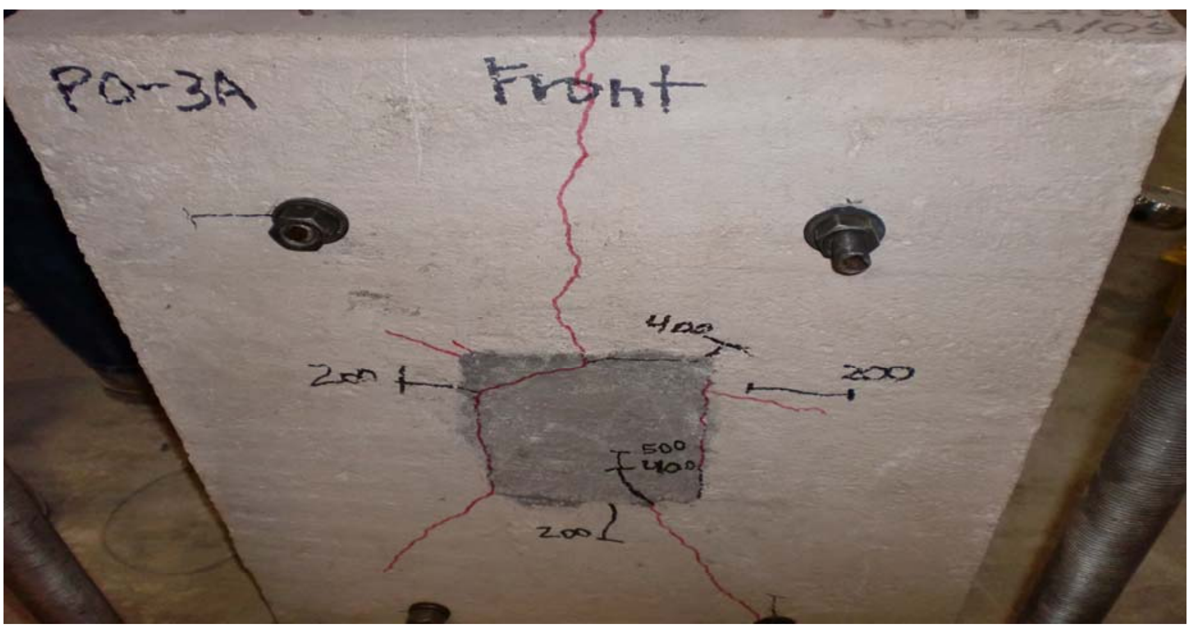


Fig. 4.13 View of the crack pattern at failure on the back side of specimen PO-3B

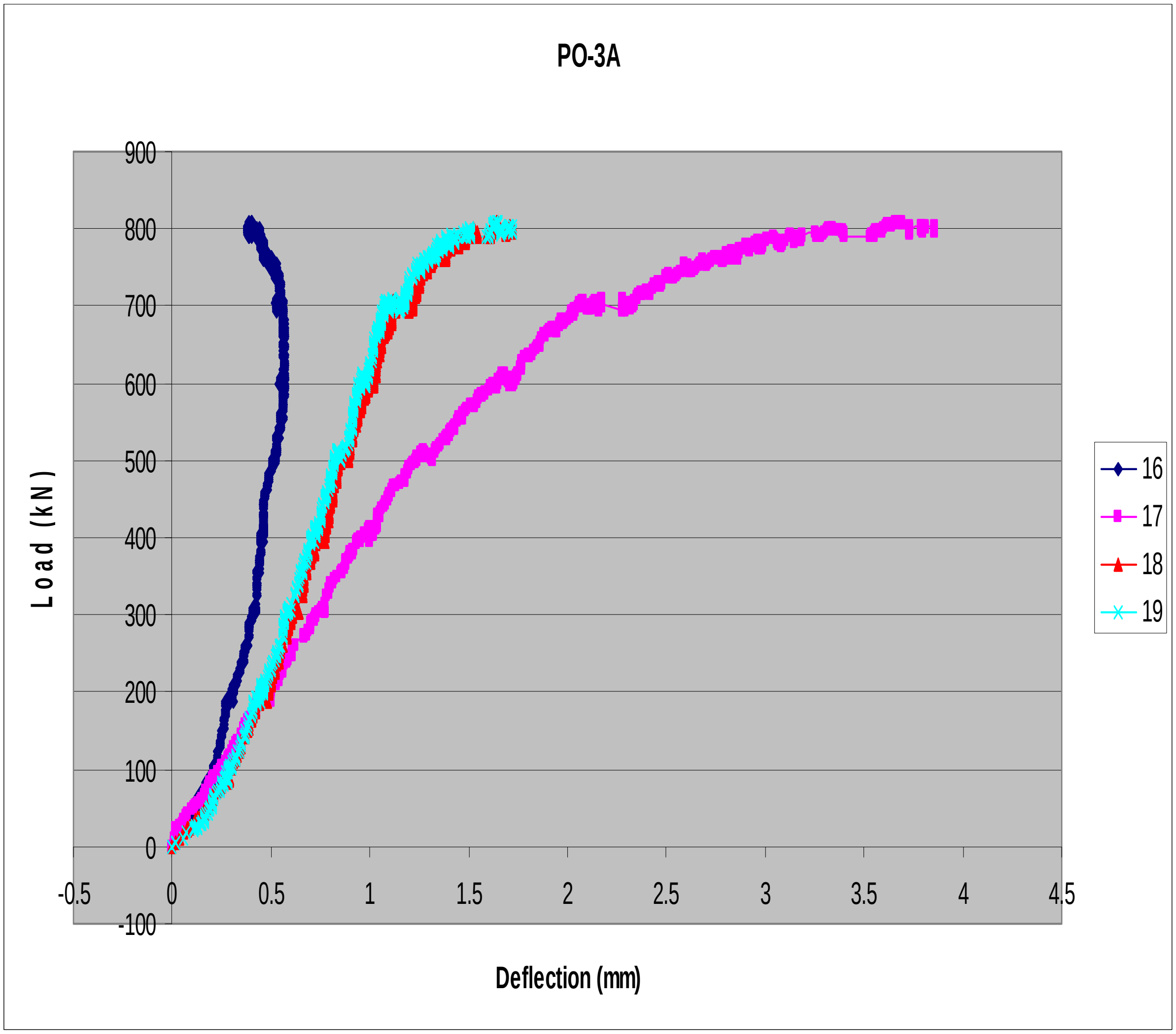


Fig. 4.14 Load-slip relationship for specimen PO-3A

In case of specimen PO-3B, Figs. 4.15 and 4.16 shows views of the crack pattern at failure on the front and back side of the specimen, respectively. It can be observed that cracks were initiated at a load of 600 kN and failure occurred at 750 kN load. Some of these crack all almost parallel to the direction of the force as a result of splitting forces in concrete. However, local crack was recorded normal and parallel to the direction of load at 600 kN load at the interface between the shear pocket and the precast slab, indicating excessive bearing and shear strength on concrete, respectively. Figure 4.17 depicts the load-slip relationship for specimen PO-3B that is similar in nature to those for specimens PO-1A and PO-1B. However, the maximum slip reached at failure was approximately 6.2 mm, with an average value of approximately 3.5 mm, given the large change in LVDT readings at failure. It interesting to mention that shear failure of the studs at the slab-shear pocket interface for specimen PO-3B was observed as depicted in Fig. 4.18.

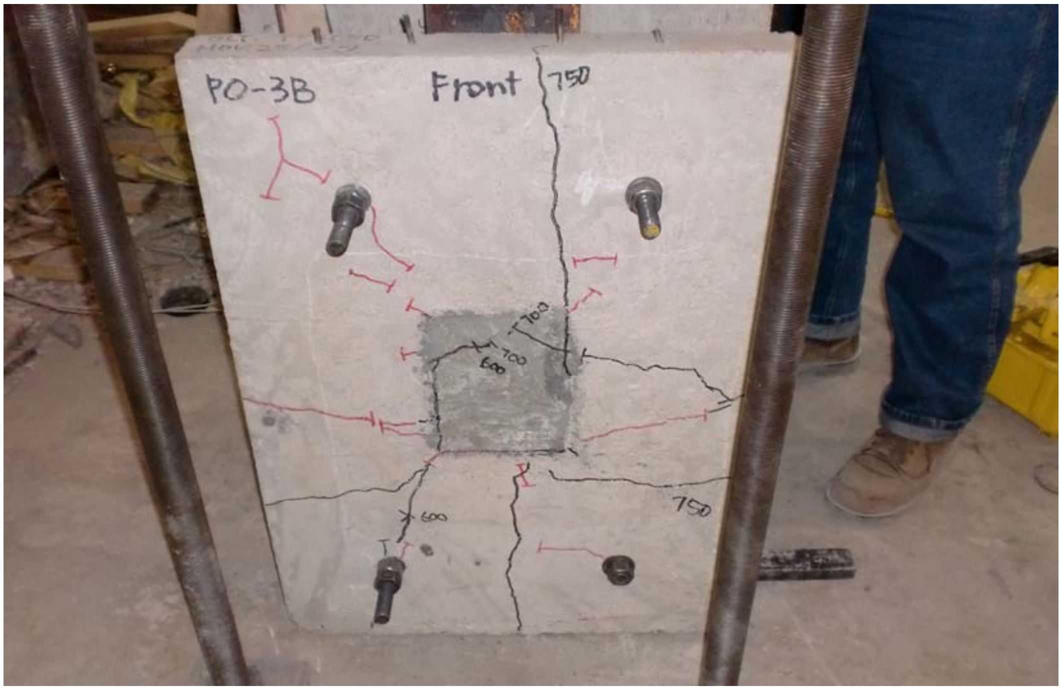


Fig. 4.15 View of the crack pattern at failure on the front side of specimen PO-3B

Fig. 4.16 View of the crack pattern at failure on the back side of specimen PO-3B

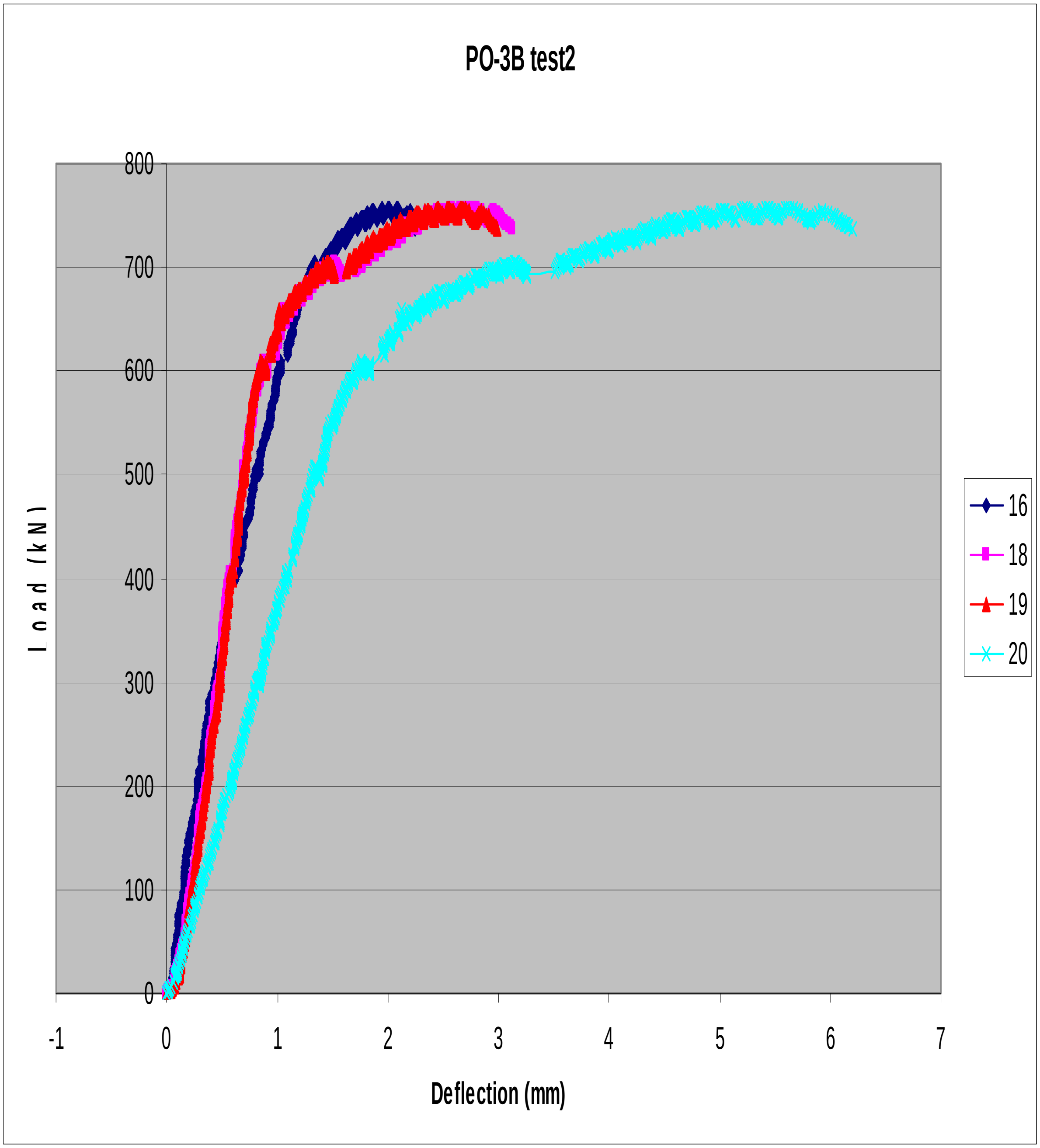
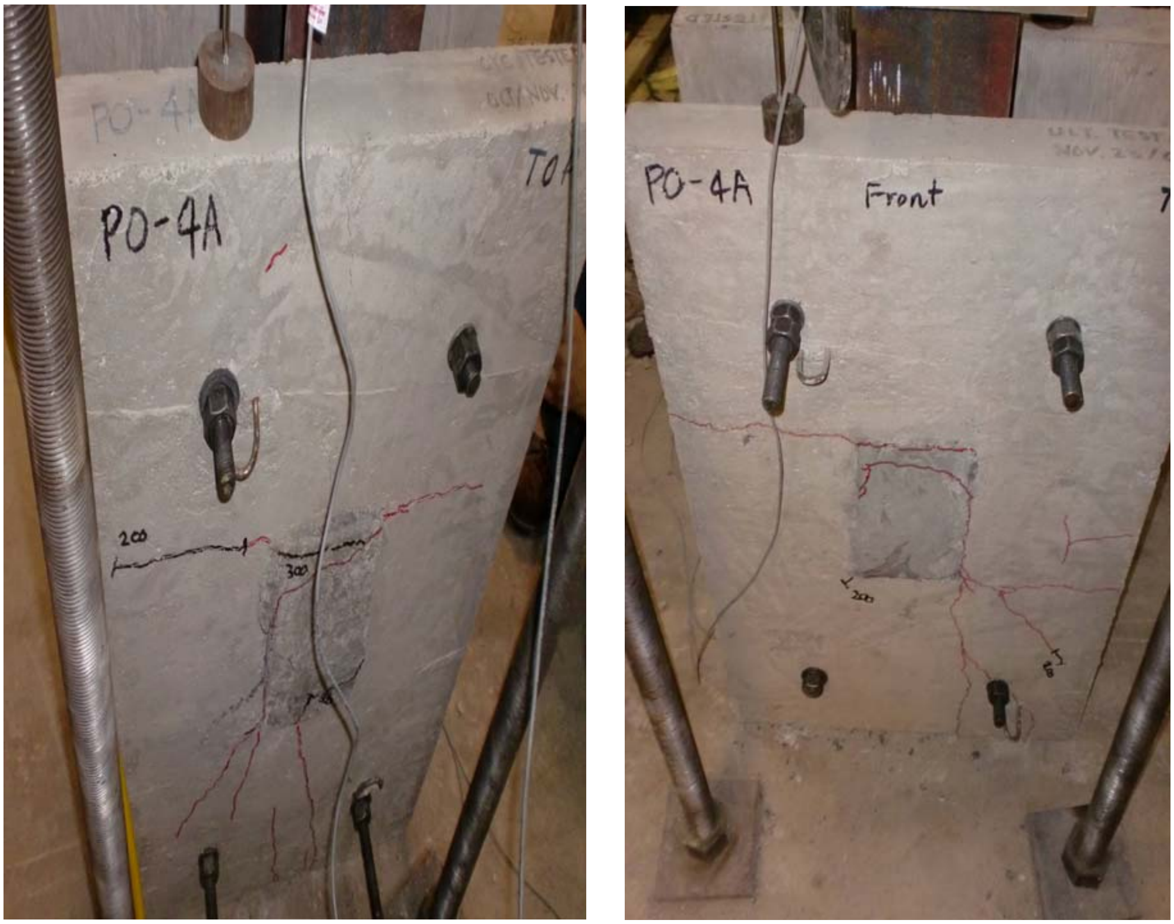


Fig. 4.17 Load-slip relationship for specimen PO-3B

Fig. 4.18 Views of the shear failure of the studs at the slab-shear pocket interface for for specimen PO-3B

4.5 Specimens with shear stud cluster with precast concrete slab and CIP concrete in the shear pocket but with no rebar projecting into the shear pocket

Specimens PO-4A and PO-4B were identical and represent the case of shear stud clusters welded to the steel beam with a precast concrete panel resting over the steel beam and a shear pocket filled with cast-in-place concrete. However, steel rebar in the precast slab were not projecting into the shear pocket to investigate possible shear pocket pop-out as a result of eliminating its connection with the preacast slab. Figures 4.18 and 4.19 shows views of the crack pattern during and at failure for specimen PO-3A. It can be observed that cracks were initiated at a load of 200 kN at the edges of the shear pocket propagating away from its ends. The failure load was recorded at 510 kN. It should be noted that cracks were recorded at the interface between the cast-in-place concrete in the shear pocket and the precast slab, leading to concrete pop-out as shown in Fig. 4.19. It should be noted that the shear pocket failure led to the split of the concrete slab away from the steel beam as shown in Fig. 4.19(c). Figure 4.20 depicts the load-slip relationship for specimen PO-4A that is similar in nature to those for specimens PO-1A and PO-1B. However, the maximum slip reached at failure was approximately 2.7 mm, with an average value of approximately 1.7 mm, given the large change in LVDT readings.



(a) Front side (b) Back side

Fig. 4.18 View of the crack pattern at failure on the front side of specimen PO-4A

(a) (b)

(c)

Fig. 4.19 Views of (a) cracks in the shear pocket near failure, (b) pop-out of the shear pocket and (c) the failure shape of damaged pocket after failure

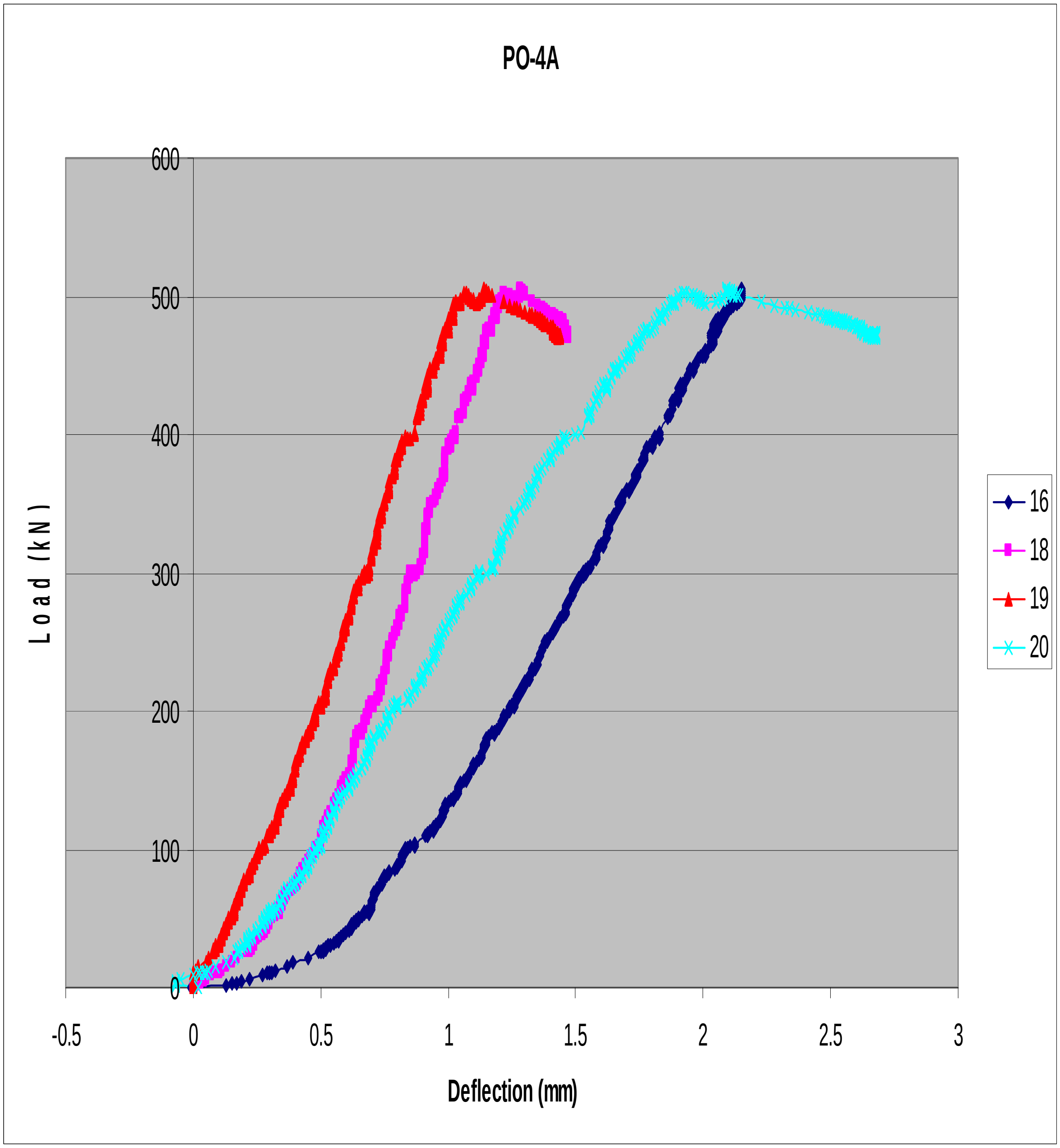
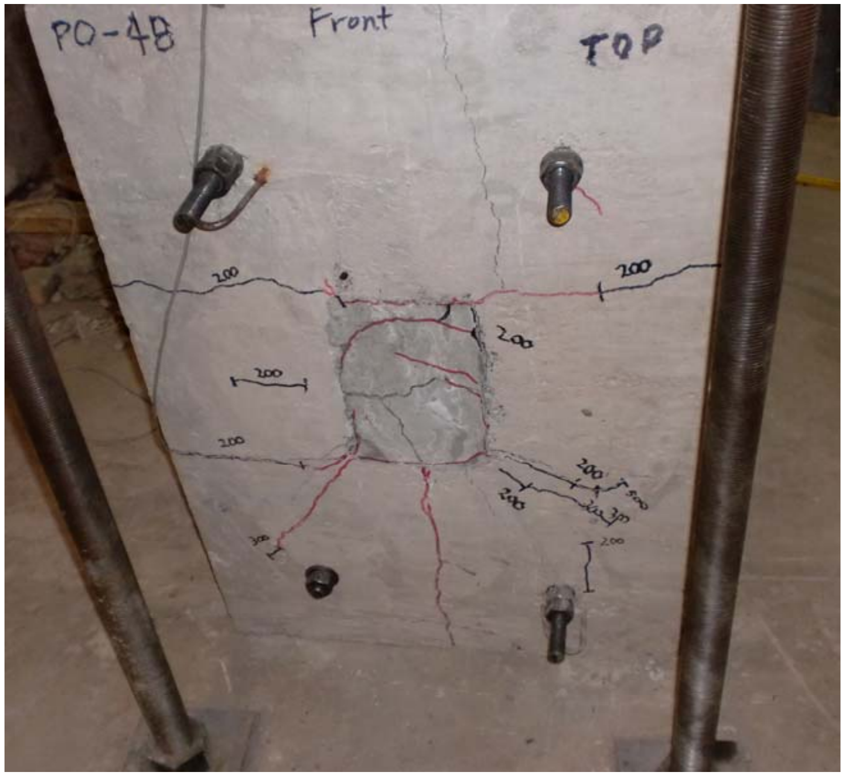
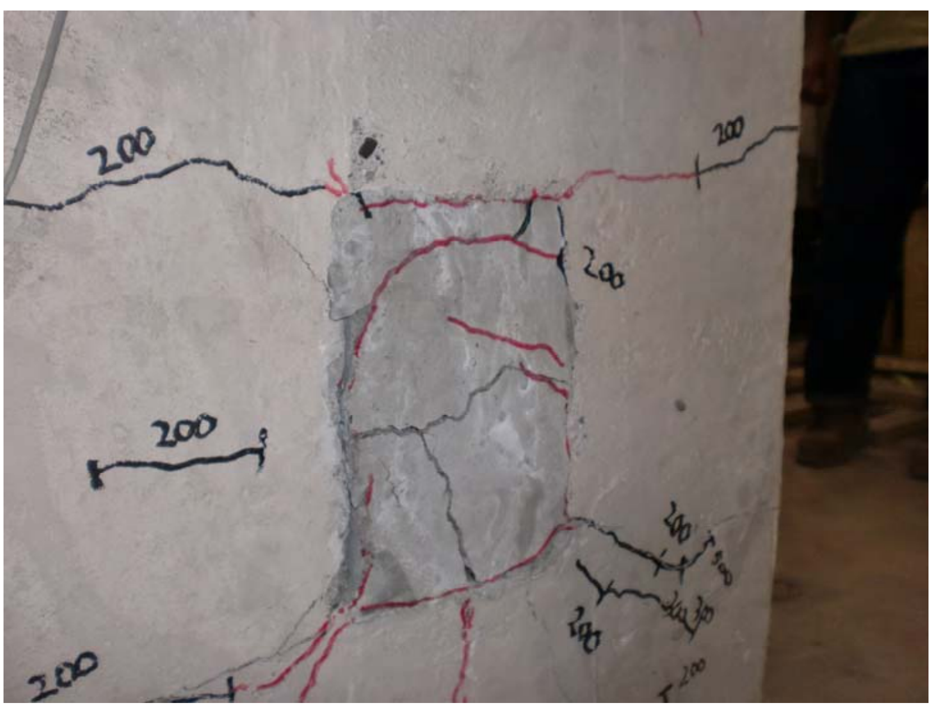


Fig. 4.20 Load-slip relationship for specimen PO-4A



(a)



(b)

Fig. 4.21 View of the crack pattern at failure on the front side of specimen PO-4B

Fig. 4.22 View of the crack pattern at failure on the back side of specimen PO-4B

Fig. 4.23 Views of the cracks and pop-out of the shear pocket of specimen PO-4B just before failure

(a) (b)

(c)

Fig. 4.23 Views of the failure mode of the shear pocket of specimen PO-4A showing portion of the CIB concrete attached to the shear cluster

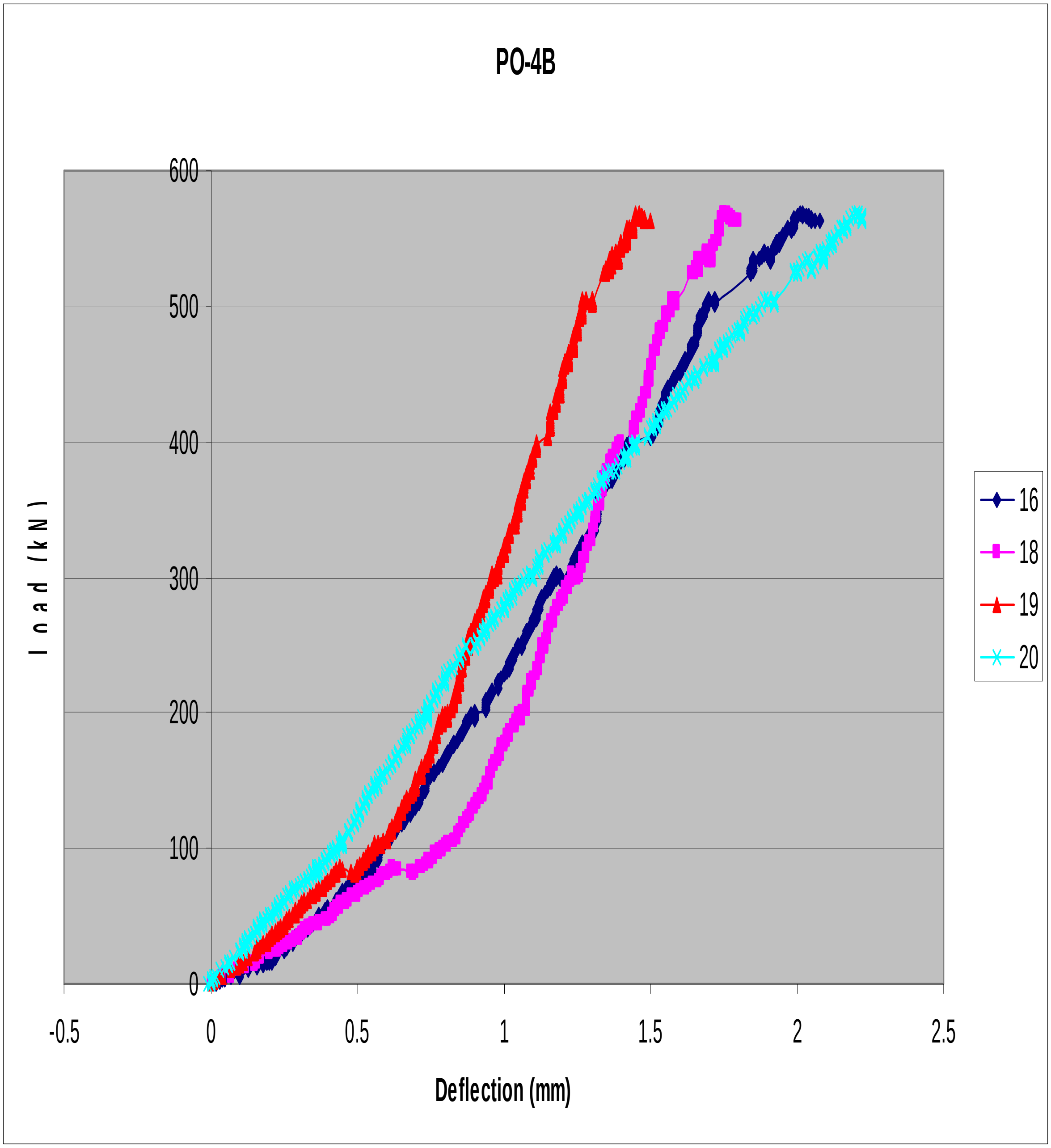


Fig. 4.24Load-slip relationship for specimen PO-4B

Similar behaviour of specimen PO-4A was observed for specimen PO-4B. Figures 4.21 and 4.22 shows the crack pattern near failure at the front and back face of specimen PO-4B, respectively. Views of concrete pop-out and shear pocket failure are presented in Fig. 4.23. While the load-slip curves are depicted in Fig. 4.24 with 2.3 mm maximum slip and 1.8 mm average slip at failure of specimen PO-4B. The ultimate load was recorded as 570 kN.

CHAPTER V

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

5.1 General

The use of prefabricated elements and systems in bridge construction has recently been the subject of much attention and interest amongst bridge jurisdictions as a way of improving bridge construction. Through mass production of the materials, the repeated use of forms, reduction of on-site construction time and labor by concentrating the construction effort in a fabrication facility rather than at the bridge site, significant economic benefits can be achieved. Aging bridges of North America may require repair, rehabilitation, or replacement. The current traditional bridge rehabilitation/replacement system in most situation is very time consuming and costly. Issues related to work zone safety and traffic disruptions are also a major concern. A full-lane closure is very costly in large busy urban highways because of the significant economic impact on commercial and industrial activities. As a result, prefabricated bridge technology is seen as a potential solution to many of these issues. Prefabricated elements and systems can be quickly assembled and could reduce design efforts, reduce the impact on the environment in the vicinity of the site, and minimize the delays and lane closure time and inconvenience to the traveling public, saving time and tax payers' money. This project investigates the full-depth precast bridge deck panels with no overlays connected to the steel girders using lumped shear connectors Eight panel-steel girder connections of different shear connector configurations were erected and tested to complete collapse to examine their structural behaviour, crack pattern and ultimate load carrying capacity. The following sections presents summary of the results and recommendations for future research.

5.2 Conclusions

Based on the data generated from the experimental study and the configuration of the tested specimens, the following conclusions are drawn:

1- The capacity of the full depth precast slab connected to the steel beam with clustered shear stud experienced an ultimate load of 780 kN as compared to 890 kN for the connection of the cast-in-place slab-to-steel beam connection with regularly spaced studs. Thus, a reduction is strength by 12 % is observed with the use of the precast panel with clustered shear studs.

2- The ultimate strength of the precast panel with clustered shear studs was 770 kN are compared to 780 kN for similar connection but with cast-in-place slab. This means that the effect of precast versus cast-in-place concrete is observed to be insignificant.

3- The capacity of the full depth precast slab connected to the steel beam with clustered shear stud experienced an ultimate load of 780 kN as compared to 540 kN for similar connection but without continuation of the steel rebar in the slab into the shear pocket. This means that the presence of the steel rebar into the shear pocket increased the connection strength by about 44%.

4- For specimens with precast panels, shear pockets experiences cracks/separation at the pocket-precast slab interface, leading to concrete pop-out.

5.3 Recommendations for Future Research

It is recommended that the following are being considered for future research:

1- Repeat the static load testing on full-size specimens under both static and fatigue loading.

2- Investigation of the use of tapered shear pockets with steel channel as shear connector.

3- 3- Conduct finite-element modelling for the push-off testing for different connection scenarios.

REFERENCES

AASHTO, "*LRFD Bridge Design Specifications*," 2nd Ed., American Association of State Highway and Transportation Officials, Washington, D. C., USA, 1998.

Abendroth, R. E., "*Nominal Strength of Composite Prestressed Concrete Bridge Deck Panels*," Journal of Structural Engineering, ASCE, Vol. 121, No. 2, 1995, pp. 307-318.

Anderson, A. R. "*How Beam Design Affects Prestressed Concrete Bridge Costs*," Eng. News-Record, October 1957, pp. 326-328.

Anderson, A. R. "*Lateral Stability of Long Prestressed Concrete Beams*," PCI Journal, Vol. 16, No. 3, May-June 1971, pp. 7-9.

Anderson, A. R. "*Systems Concepts for Precast Prestressed Concrete Bridge Construction,*" Special Report 132, Highway Research Board, Washington, DC, 1972, PP. 9-21.

Anderson, A. R. "*Stretched-Out AASHTO-PCI Beams, Type III and IV, for Longer-Span Highway bridges*," PCI Journal, Vol. 18, No. 5, September-October 1973, pp. 32-49.

Anderson, A. R. "*Prefabricated Bridge Elements and Systems*," NCHRP Synthesis 119, Project 20-5 FY 1983, National Cooperative Highway Research Program, August 1985.

Applied Technology Council, "*Proceedings of a Workshop on Design of Prefabricated Concrete Buildings for Earthquake Loads*", ATC-8, Applied Technology Council, Berkeley, California, 1981.

Arockiasamy, M., Badve, A. P., Rao, B. V. and Reddy, D. V. "*Fatigue Strength of Joints in a Precast Prestressed Concrete Double Tee Bridge*", PCI Journal, Vol. 36, No. 1, January-February 1991, pp. 84-96.

, Vol. 5, No. 2, 2000, pp. 139-147.

Bakht, B. and Mufti, A. A., "*Load Distribution in Shear-Connected Concrete Plank Bridges,*" 29th Annual Conference of the Canadian Society for Civil Engineering, Victoria, British Columbia, Canada, 2001a.

Bakht, B., Mufti, A. A. and Sargent, D. D., "*Assessment of Stresses in Welds of Shear-Connected Concrete Plank Bridges,*" 29th Annual Conference of the Canadian Society for Civil Engineering, Victoria, British Columbia, Canada, 2001b.

Burns, N. H. and Centennial, Z., "*Composite Action between Girder and Bridge Deck with Precast Panels*", PCI Journal, Vol. 46, No. 1, 2001, pp. 85.

Canadian Standard Association, "*Canadian Highway Bridge Design Code CAN/CSA-S6-00*," CSA International, Toronto, Ontario, Canada, 2000.

Curtis, R. B. "*Single-T Bridges*," PCI Journal, Vol. 12, No. 2, March-April 1967, pp. 76-81.

Dolan, C. W. and Pessiki, S. P., "*Model Testing of Precast Concrete Connections*," PCI Journal, Vol. 34, No. 2, 1989, pp. 84-103.

Ellobodya, Ehab. 2005. Performance of shear connection in composite beams with profiled steel. Department of Structural Engineering, Faculty of Engineering, Tanta University, Tanta, Egypt.

Geren, K. Y., and Tadros, M. K. "*The NU Precast Prestressed Concrete Bridge I-Girder Series*," PCI Journal, Vol. 39, No. 3, May-June 1994, pp. 26-39.

Hariatmadar, H., "*Seismic Response of Connections in Precast Concrete Double Tees*," PhD Dissertation, Department of Civil Engineering, McGill University, Canada, 1997.

Hofheins, C. L., Reaveley, L. D. and Pantelides, C. P., "*Behavior of Welded Plate Connections in Precast Concrete Panels under Simulated Seismic Loads*," PCI journal, Vol. 47, No. 4, 2002, pp. 122-133.

Kim, Boksun. 2001. The behavior of through-deck welded shear. Scott Sutherland School of Architecture, Robert Gordon University.

Kwei, G. C. S. "*Discussion of 'Single T-Beam Bridges*,' by R. B. Curtis," PCI Journal, Vol. 12, No. 5, September-October 1967, pp. 82-83.

McKeel, W. T., "*Bridge Maintenance and Management: A Look to the Future*," A3C06: Committee on Structures Maintenance and Management, Transportation Research Board, National Research Council, Washington, D.C., USA, 2002.

Mufti, A. A., and Newhook, J. P., "*Punching Shear Strength of Restrained Concrete Bridge Deck Slabs,"* ACI Structural Journal, Vol. 95, No. 4, 1998, pp. 375–381.

PCI Committee on Connection Details, "*Addendum to Design and Typical Details of Connections for Precast and Prestressed Concrete* ", PCI journal, Vol. 40, No. 5, 1995, pp. 46-57.

PCI Committee on Connection Details, "*Standard Precast Connections*", PCI Journal, Vol. 43, No. 4, 1998, pp. 42-58.

Pincheira, J. A., Oliva, M. G. and Kusumo-Rahardjo, F. I., "*Tests on Double Tee Flange Connectors Subjected to Monotonic and Cyclic Loading"*, PCI Journal, Vol. 43, No. 3, 1998, pp. 82-96.

Roller, J. J., Russell, H. G., Bruce, R. N. and Martin, B. T., "*Long-Term Performance of Prestressed, Pretensioned High Strength Concrete Bridge Girders*," PCI journal, Vol. 40, No. 6, November-December 1995, pp. 48-59.

Shahaway, M. E. "*Feasibility Study of Transversely Prestressed Double-T Bridges*," PCI Journal, Vol. 35, No. 5, September-October 1990, pp. 56-69.

Shahaway, M. E., and Issa, M. "*Load Testing of Transversely Prestressed Double-T Bridges*, "PCI Journal, Vol. 37, No. 2, March-April, 1992, pp. 86-99.

Tadros, M. K., Badie, S. S. and Kamel, M. R., "*Girder/Deck Connection for Rapid Removal of Bridge Decks*," PCI journal, Vol. 47, No. 3, 2002, pp. 58-68.

Yamane, T., Tadros, M. K., Badie, S. S. and Baishya, M. C., "*Full Depth Precast, Prestressed Concrete Bridge Deck System",* PCI journal, Vol. 43, No. 3, 1998, pp. 50-66.