SHEAR CONNECTORS FOR STEEL-PRECAST COMPOSITE BRIDGES

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BIOGRAPHY

Scott Walbridge is an Associate Professor at the University of Waterloo. Prior to starting there in 2006, he obtained engineering degrees from the University of Alberta and Ecole Polytechnique Federale de Lausanne and worked as a structural consultant for several years in Edmonton, Canada. Scott's research investigates various aspects of metal bridge design, assessment, and rehabilitation. In particular, his research interests include: fatigue assessment and retrofitting, aluminum use in bridges, and steel-precast shear connections for rapid bridge construction.

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SUMMARY

Currently, the poor condition of many Canadian bridges warrants complete replacement as they are obsolete or deteriorated to the point such that rehabilitation is ineffective in further extending their service lives. Modular bridge systems consisting of steel girders and precast concrete slabs have become increasingly popular for bridge replacement under these circumstances.

The shear connections employed for systems can significantly impact the construction time, economic and environmental cost, structural integrity, and durability of the bridge. The most common connections involve steel studs that are welded to the girder top flanges in groups coinciding with grouted pockets, rather than spaced continuously along the span. While this approach is simple, it does not result in a rapid or durable connection, as copious small quantities of grout must be poured into the pockets, slowing assembly and resulting in cold joints in the concrete.

This paper presents: 1) a brief review of the state-of-the-art in research on alternative shear connection solutions for steel-precast composite bridges, 2) an overview of the research recently conducted at the University of Waterloo on: i) panel end shear connectors, ii) through bolt shear connectors, and iii) connections employing match cast shear lugs, and 3) a discussion of gaps in the state-of-the-art that need to be addressed in order to improve the constructability and durability of modular steel-precast composite bridges.

SHEAR CONNECTORS FOR STEEL-PRECAST COMPOSITE BRIDGES

Introduction

Currently, the poor condition of many Canadian bridges warrants complete replacement as they are functionally obsolete or deteriorated to the point such that rehabilitation is ineffective in further extending their service lives. Municipal and provincial bridge authorities across the country are looking to replace the structures while minimizing the impact that construction has on the transportation network. Modular bridge systems consisting of steel girders and precast concrete slabs have become increasingly popular for bridge replacement under these circumstances. The benefits offered by these structures include the possibilities of exploiting the advantages of both materials to create economic, lightweight structures and reducing construction time by prefabrication of the main structural elements.

The shear connections employed for such modular systems can significantly impact the construction time, economic and environmental cost, structural integrity, and durability of the bridge. The most common shear connections involve steel studs that are welded to the girder top flanges in groups rather than spaced continuously along the span, as is normally done for cast-in-place slabs. Full-depth pockets are provided in the precast panels to coincide with the stud group locations. Once the panels are placed on the girders, the pockets are grouted to create the shear connection. While this connection approach is simple, it does not result in a rapid, durable shear connection, as the copious small quantities of concrete that need to be poured into the pockets slow down assembly and result in vertical cold joints in the concrete.

This paper presents: 1) a brief review of the state-of-the-art in research on alternative shear connection solutions for steel-precast composite bridges, 2) an overview of the research recently conducted at the University of Waterloo on: i) panel end shear connectors, ii) through bolt shear connectors, and iii) connections employing match cast shear lugs, and 3) a discussion of gaps in the state-of-the-art that need to be addressed in order to improve the constructability and durability of modular steel-precast composite bridges.

State-of-the-art Review

In the following paragraphs, the state-of-the-art in research on shear connections between full-depth precast concrete deck panels and steel girders is briefly reviewed. This review is divided into the following sub-sections: 1) pocketed stud groups, 2) post-installed inserts and through-bolt shear connectors, and 3) connection by adhesion.

The first use of full-depth precast concrete bridge deck panels occurred in the 1970s at several locations in the United States, however, these bridges were primarily non-composite [1]. Advances were made to precast technology and a decade later transportation departments were using composite designs, realizing the advantages of having a proper shear connection. Although it varies with bridge geometry, a composite deck and girder assembly will increase strength by 20-50% when compared to a bridge with no interaction [2]. The shear connection in modular precast systems may also have a significant impact on constructability, cost, and durability [3].

Pocketed Stud Groups

In steel-precast composite bridges, the shear studs are commonly group together at discrete locations so that the precast deck panels can be attached to the girders by providing full depth “shear pockets” filled with grout. Typically these pockets feature 4-8 studs per group or “cluster” and are spaced at 20”-40” (500-1000 mm) on centre. The Canadian Highway Bridge Design Code (CAN/CSA S6-06) [4] did not address grouted shear pockets for full depth precast panels until recently, but now provides guidelines based on research conducted in Canada and the US since 2005. An example of shear stud clusters is shown in Figure 1.

Despite the widespread use of grouted shear pockets in steel precast composite bridges, they impose several disadvantages to the system. From a constructability standpoint, the number of connections can become copious on long multi-span bridges, and the grout requires time to cure. The result is increased construction time and the presence of vertical cold joints in the concrete [5]. Additionally, the corners of the shear pockets promote stress concentrations, sometimes resulting in cracking of the concrete deck between the pockets [6]. Furthermore, the grouted connections do not facilitate quick and clean dismantling of the bridge deck and thus impede demolition during repair or decommissioning of the bridge.

Figure 1: Shear stud clusters [7].

The effect of stud clusters has been researched in Canada and the United States, primarily with the intent of increasing the current code limitations. Increased spacing of shear stud groups will result in increased economy at the fabrication and construction stages, while increasing the amount of space for the layout of transverse slab reinforcement and reducing the possibility of water leakage between the grouted perimeters [8]. Alleviating water leakage problems will help to improve the durability of the precast deck system.

Several American National Cooperative Highway Research Program (NCHRP) reports, written by Tadros and others, have been integral to the body of research surrounding the rapid replacement of bridge decks [9] and the use of pocketed stud groups [8]. Tadros and Badie conducted push-out tests and beam tests with the goal of showing that the stud spacing limit for grouped shear studs could be doubled. They propose that 48" (1200 mm) is adequate. Their experimental program showed that with an increase of about 25% more shear studs, clusters can be placed at 48" (1220 mm) with no modifications to the fatigue design procedure [8].

The Ministry of Transportation of Ontario (MTO), conducted an experimental program to study the effects of shear stud clusters in composite girder design. Huh et al. [10] assembled scaled composite beams with shear stud clusters spaced at ~16" (400 mm) and ~24" (600 mm), in addition to a control specimen with conventional uniformly spaced studs. Although push-out tests were also conducted, the testing of beams is particularly valuable from a research perspective. The beams were tested in cyclic loading followed by static loading until failure. Based on six composite beam tests and eight push-out tests, Huh et al. concluded that shear stud clusters can provide full composite action and stud clusters have little impact on the fatigue and ultimate strength characteristics of the steel-precast composite system [10].

Research at the University of British Columbia conducted by Elwood and LaRose in 2006 addressed the lack of code provisions for shear stud clusters at that time. Nine push-out tests were loaded to failure monotonically, and six tests were loaded in fatigue for up to 750,000 cycles, following by monotonic loading until failure. For the fatigue testing, a unidirectional sinusoidal wave was used for the loading at a frequency of 1 Hz. While some researchers have reported that the ultimate capacity of clusters is not linearly related to the amount of studs per cluster [1], Elwood and LaRose found this assumption of linearity to be roughly accurate [11]. In addition to the push-out tests, a bridge model was created using the structural analysis program STAAD.Pro to evaluate deflections for several stud cluster spacings based on the stiffness obtained in the push-out tests, and also assuming theoretically rigid stiffness. The results of their study caused Elwood and LaRose to recommend extending the ~24" (600 mm) code spacing limitation to ~48" (1200 mm). It was concluded that the ultimate strength equations provided in CAN/CSA S6-06 represent a lower bound for the test results. Although they noted a reduction in static shear strength with increased fatigue loading, they did not have enough data to predict the residual strength and recommended further study.

The AASHTO Bridge Design Specifications [12] do not provide any guidelines for the spacing of shear stud groups, but instead the maximum spacing falls under the limitation of 24" (610 mm) for shear studs in general [8]. This is a limit assigned by Slutter and Fisher to prevent the separation of the slab and the steel beam, with no experimental or analytical derivation given [1]. Recognizing the work of researchers including Tadros and Badie (2008), Elwood and LaRose (2006), and Markowski (2005), CAN/CSA S6-06 has made provisions for grouted pockets with shear stud clusters, and extended the spacing limit to ~48” (1200 mm). If the spacing exceeds ~24” (600 mm), the minimum number of studs per cluster is varied according to the variation in interface shear. The use of studs with a diameter greater than 1” (25 mm) is not permitted.

Post-Installed Inserts and Through-Bolt Shear Connectors

The motivations for moving away from grouted stud pockets include: the high costs of procuring and installing the high strength grout, waiting for the grout to cure, durability issues associated with having varying deck concrete strengths, and the difficulty of dismantling the grouted panels. “Design for deconstruction” is a feature that forward-looking agencies should be considering, and the use of post-installed shear inserts and/or high strength through-bolt shear connectors seems ideally suited for this purpose.

High Strength Bolt (HSB) connectors may have advantages when considering fatigue, durability, and the amount of time before a constructed bridge can be serviceable. These advantages are due to the absence of grout and welds. HSB connections have been researched since the 1970s. Much of the research has been focused on post-installed HSB shear connectors with the intent of developing composite action in non-composite bridges.

Different concepts involving bolt connectors have been proposed, but most fall into one of two distinct classes; some bolt connectors are embedded in the concrete (at the precast facility, or through adhesion in drilled holes in the case of post-installed connectors), and others pass through holes the full depth of the concrete deck. Examples of post-installed (embedded) inserts and through-bolt connectors are shown in Figure 2.

The embedded bolts behave similarly to stud shear connectors. Through-bolts initially carry shear through friction between the slab and girder flange. Through-bolts will also carry shear through bearing when slip occurs. Tests have shown that shear connectors without welds perform much better in fatigue than those requiring welds, and the pretensioning of through-bolt connections may even further reduce the fatigue effects [13]. Through-bolt and embedded bolt connections have been demonstrated in practice on the Amsterdam Interchange Bridge rehabilitation project in New York and the Conestogo River Bridge in Waterloo. In the former project, inspectors provided positive reviews [6], citing that the decks with bolted connections performed as well as cast-in-place decks (the bridge features both deck types).

Disadvantages of the through-bolt system include the pretensioning of many individual connectors on site and the elevated level of precast quality control required in order to ensure that the holes in the deck line up with the holes drilled in the girder flanges. Flange holes could be drilled on site to ease these tolerances, but this approach has its own drawbacks. Still, the avoidance of grouting, improved fatigue performance of bolts, and ease of deconstruction could mean that through-bolts are a viable alternative to welded shear studs.

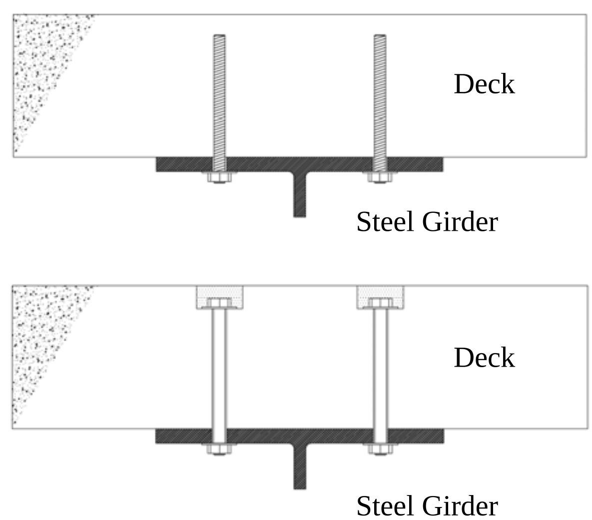

Figure 2: Post-installed inserts (top) and through-bolt shear connectors (bottom).

Research into high strength through-bolt connections has been minimal compared to that of stud shear connectors. In 1979 Rabbat and Hanson investigated the performance of through-bolt connections between precast concrete deck slabs and supporting girders [13]. They used the shear-friction procedure of the 1971 ACI Building code to estimate the ultimate capacity of the connection.

This shear-friction procedure is very similar to that found in the Canadian Concrete Code (CAN/CSA A23.3) [14], where it is referred to as "interface shear transfer". Static tests had been performed on bolted connections prior to 1979, but no data existed concerning fatigue, and specifically high-cycle fatigue [13]. Rabbat and Hanson were performing their research for the Metropolitan Atlanta Rapid Transit Authority, looking for a connection detail for an elevated transit structure to accelerate construction while minimizing traffic disruptions. Using a modified push-out test, steel-precast bolted connections were cycled 2 million and 5 million times at 3 different load ranges. It was confirmed that cyclic loading reduced the strength of the bolts, in some cases as much as 14%. Prestressing of the bolts tended to minimize the negative effects of the cyclic loading by reducing the stress ranges in the bolts.

MTO has also performed tests on a through-bolt connection system. Au et al. (2010) studied the ability of a bolted connection detail to induce composite action, withstand cyclic loading, and to perform at comparably high ultimate loads in flexure to conventional shear connection systems [15]. Two 13'-9" (4200 mm) long beams were fabricated and tested, each with 38 pairs of bolts at a 4.3" (110 mm) spacing, and then compared to a control specimen with conventional stud shear connectors. 3 million cycles were applied to each beam specimen. Au et al. found that although the level of composite action achieved was not as strong as for the control beam, it was "reliable and effective" [15]. In their tests, cyclic loading did not affect the performance of the system, and the ultimate flexural resistance of the beam exceeded the theoretical CAN/CSA S6-06 value.

Kwon et al., out of the University of Texas at Austin, have published several articles on the behavior of post-installed shear connectors. They have examined the viability of using through-bolts (termed "high-tension friction grip bolts" in their testing) and two other bolted connections to strengthen existing non-composite bridges by developing composite action. Full-scale beam tests, direct shear tests (analogous to a horizontal half push-out test), and even field implementation were used to assess the performance of the bolted connections. The direct shear tests did result in fatigue failures of the bolts. The high-tension friction grip bolt in particular failed after 5 million cycles at a stress range of 240 MPa (nominal shear stress in the bolt) – far higher than the welded shear stud AASHTO [12] equation predicted.

Kwon et al. noted that the high fatigue strength of the bolts allowed a lower number of connectors to be used (relative to the number of shear studs needed). The field implementation portion of the work saw a bridge near Hondo, Texas gain a 65% increase in load rating [16]. It is worth noting that Kwon et al. developed an equation for the fatigue endurance limit of an embedded adhesive bolted connection, but did not have enough data to do the same for the through-bolt connection used.

There are currently no code provisions for using high strength bolts in lieu of stud shear connectors in composite bridge construction. Although the CAN/CSA S6-06 Code Commentary mentions a successful pilot project conducted in 1977 near Waterloo, Ontario using bolted shear connections, the bolts used were embedded in the deck, and not the through-bolt type connection [17].

Connection by Adhesion

Lebet and Thomann [5] studied the use of an embossed vertical plate, welded to the girder top flange, to create the steel-precast shear connection. According to this approach, the precast panels contain a longitudinal groove, slightly larger than the plate. Prior to installation of the precast deck, the surfaces of the panels that will come in contact with the girder top flange are roughened, with an amplitude of at least ~0.25" (6 mm), to enhance the bond. The use of a surface applied chemical retarder is suggested as an option for the surface roughening. The prefabricated panels are slid along the girders into place. Grout is then injected into the notch to create the shear connection. Along with the use of embossing to strengthen the connection between the plate and grout, they also studied the use of a polymer-aggregate bonding layer between the panel and the girder.

Direct shear push tests of several variations of this connection type have been conducted [18]. For comparison, push tests were also completed on a traditional shear connection consisting of headed studs. Following the completion of the push tests, a mechanical model for connection "by adherence" was developed to determine if the limited ductility of the embossed plate connection could present a problem in composite beams and to determine how the geometry of the steel-concrete interfaces affects the performance of the connection with particular attention to the influence of normal stress on shear resistance. Despite the low ductility, it is shown in [5] that composite beams with this connection type can provide full plastic flexural resistance in both positive and negative bending.

Ramsay [19] from the University of Toronto conducted a series of push tests to investigate the suitability of a polyurethane interface to bond full-depth precast concrete deck panels to steel girders. Four different joint configurations were tested in this study, including a variety of soffit and embedded steel plate configurations. The results showed cohesive failures of the polyurethane at lower than expected stress levels.

A subsequent study was undertaken by Cheung [20] to investigate the behavior of the polyurethane. Cheung tested six formulations of polyurethane using a scaled down version of the push test employed in [19]. Based on this work, a polyurethane formulation was found that should provide adequate strength and stiffness for use in composite bridges with full depth precast concrete decks. Further laboratory testing is recommended, however, prior to field application.

Recent Research Conducted at the University of Waterloo (UW)

In the following paragraphs, recent research conducted at UW on shear connectors for steel-precast composite bridges is briefly reviewed. This review is divided into the following sub-sections: 1) panel end shear connectors, 2) through-bolt shear connectors, and 3) shear connections employing match cast shear lugs.

Panel End Shear Connectors

For his master's thesis project, Bowser [21] performed a multi-criteria assessment of ten shear connection concepts. After identifying the panel end shear connection as the leading shear connection method, he created a Finite Element (FE) model to study the effects that the connections had on bridge girders. A 118' (36 m) span twin girder bridge was designed using code provisions, and the panel end shear connection was modeled in ABAQUS. A panel end connection, as proposed by Bowser, consists of a gusseted bearing plate welded to the girder top flange, which is bolted to a precast panel steel embed assembly at the panel ends. The embed assembly is a longitudinal frame with shear studs along its length, placed in the formwork at the precast plant. Block-outs are provided at the ends of the precast panel so that wrenches or impact guns can be used to fasten the gussets to the faces of the steel embed assembly.

The load pattern in Figure 3 corresponds to the critical position of the design truck for flexure on the 118' span. After proving that flange buckling issues could easily be prevented, Bowser studied the effect of the panel end connection spacing on the response of the bridge. It can be seen in Figure 3 that as the spacing of the connectors increased, the maximum applied moment and stiffness of the bridge decreased, while the mid-span deflection increased. Although most precast panels do not exceed 9.8' (3 m), it is interesting to note that even with a spacing of 19.7' (6 m), the response of the bridge changes negligibly. A similar parametric study was completed on the individual connection stiffness.

Bowser's researched confirmed that possible local effects (e.g. compression flange or web buckling) due to the use of precast panel end connections did not negatively influence the performance of the composite bridge system. Bowser's parametric studies on the connector stiffness and spacing, combined with his multi-criteria assessment and model verification, show the panel end connection system to be constructible and capable of providing the same structural response as a conventional shear stud system [21].

In a subsequent master's thesis project, Chen [22] continued work on Bowser's panel end connection FE model, analyzing it for load-displacement behavior, cross-sectional stress and strain profiles, and connection force distributions. Chen analyzed the behavior of the panel end connection system when compared to a purely composite model, a shear stud model, and a model with discrete connectors 10 times less stiff than the proposed panel end connection. Examining strain profiles through the composite girder depth, Chen found that composite action was strong in the panel end connection model, and slightly decreased with the more flexible connection. The ultimate moment capacity of the panel end connection system was only 1.5% less than that of the purely composite girder, and deflections were only 3% larger [22]. While these results show promise, static tests on beams are a recommended next step for studying potential constructability hurdles.

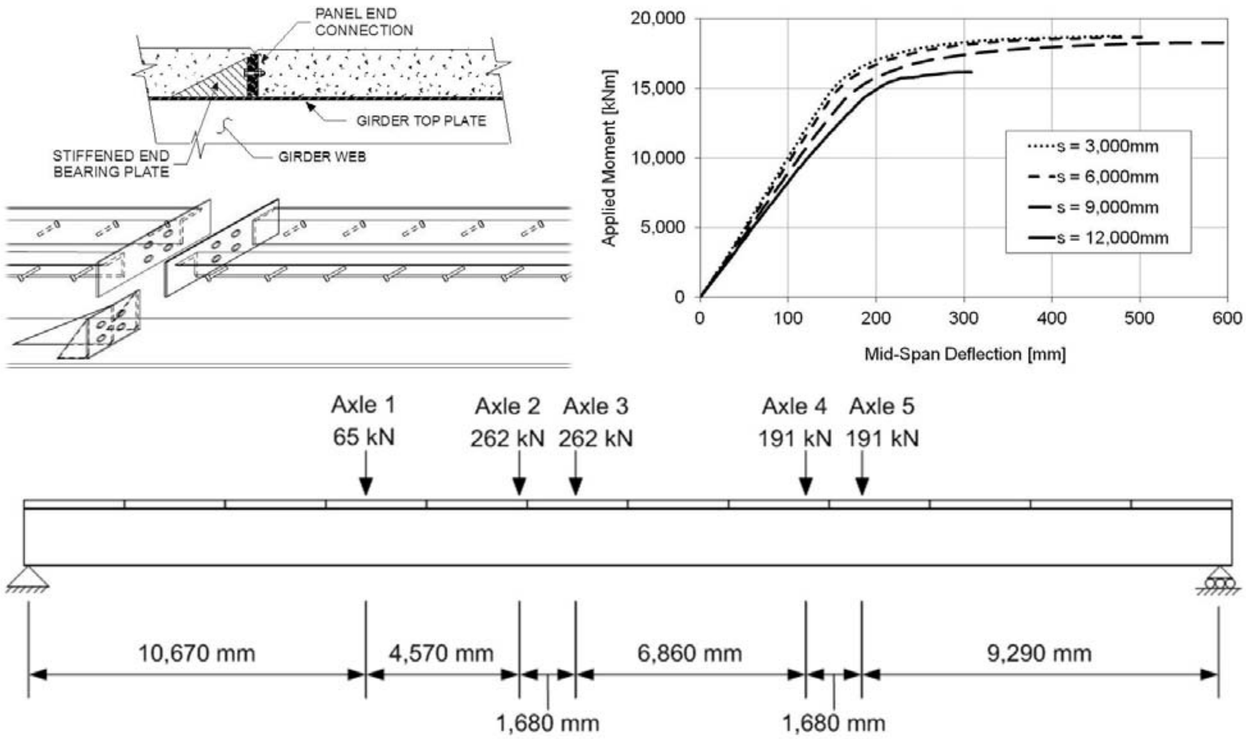

Figure 3: Panel end shear connection and FEA results for full-scale bridge analysis [21].

Through-Bolt Shear Connectors

In addition to extending the analytical work of Bowser on panel end connections, Chen conducted an experimental program comparing the results of push tests using through-bolts [22]. In some cases these specimens also contained match-cast shear lugs (as discussed in the next section). Parameters varied included bolt diameter, bolt pretension, and the friction properties of the steel-concrete interface. Through-bolt push test results were compared with a control specimen made with standard shear studs. Figure 4 shows the push test geometry (with dimensions in mm), as well as a photo of a specimen inside the test frame.

The slip loads and the peak loads of the through-bolt connected specimens were lower in many cases than the conventional stud shear connected specimen, although direct comparison is not straight forward due to the varying connector diameter. Overall, the bolts exhibited a high initial stiffness and ductility, but a low initial slip load. Figure 5 shows two plots of applied load vs. measured slip for one steel-concrete interface condition (CS2). These plots show the reduced capacity of the bolted connections compared to the stud shear connection (load, *P*, per connector is plotted), as well as the positive effect of increasing the bolt diameter and/or pretension.

Chen developed a simple mechanistic model to estimate the ultimate capacity of the through-bolt connection based on 13 push-out test results. It was observed that the ultimate shear capacity of the bolted connection is the sum of the friction resistance (based on the shear friction procedure from [13]) and the bolt dowel action resistance. An ultimate capacity mechanism was proposed, which takes into account the bolt incline at failure and interaction between shear and tension to determine the level of dowel action.

Following Chen's research, Zhao incorporated the load-slip curves from the push-out tests in FE models of full-scale bridge girders to confirm that ductile girder behavior can be achieved, even with the low initial slip load (see Figure 6).

Match Cast Shear Lugs

On several of Chen's specimens, steel blocks or "shear lugs" were welded onto the top flange of the beam and the slab was match cast, so that it could be placed directly over the shear lug during installation, prior to bolting. It was expected that this would result in a significant static strength increase, as the shear failure can only occur in this case if there is a bearing failure in the concrete, or a failure of the through-bolts or the weld between the shear lug and girder flange.

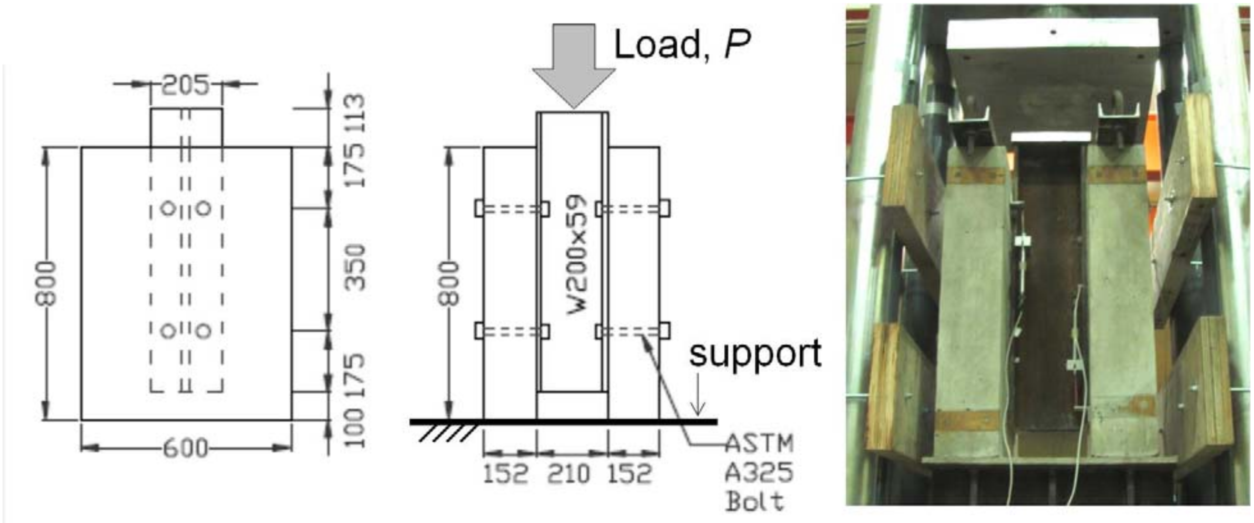

Figure 4: Push tests on through bolt shear connectors (dimensions in mm) [22].

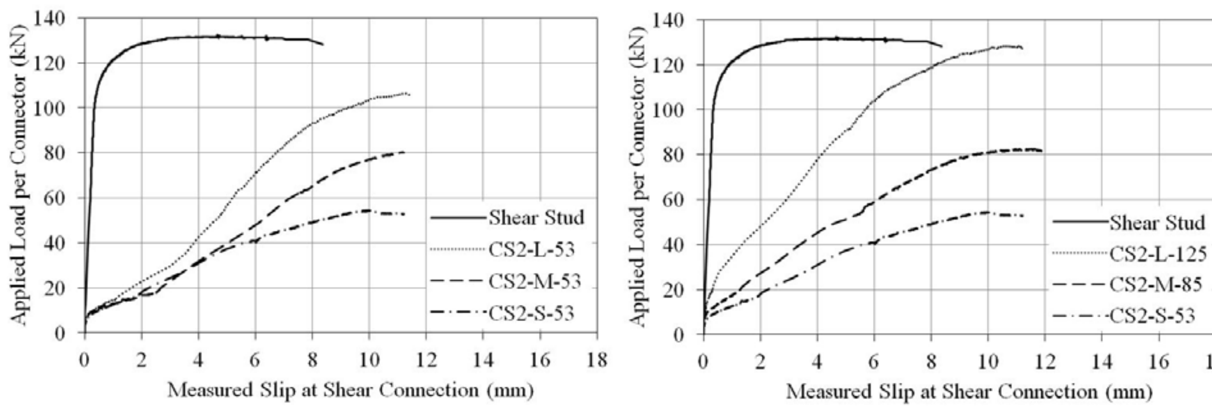

Figure 5: Load-slip results for three bolt sizes: pretension = 53 kN (left) and 70% of F_u (right) [22].

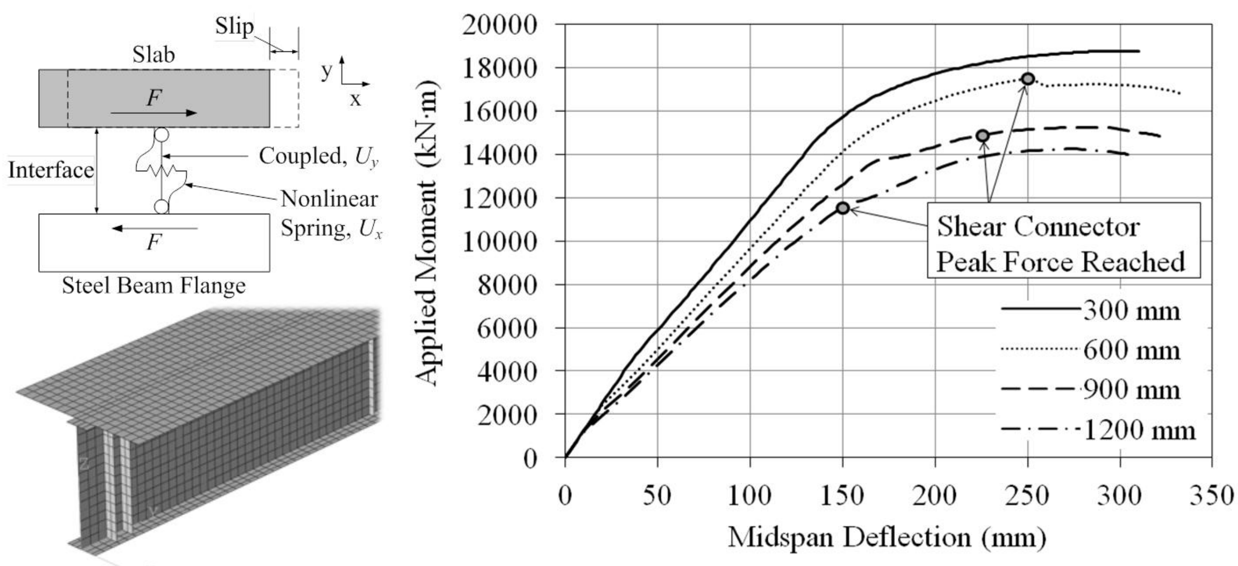

Figure 6: FE analysis of full-scale bridge girder with 2 x 16 mm through-bolts at various spacings.

Figure 7 shows one slab of a shear lug push-out test specimen after failure. The slab has been cut in two pieces for inspection and the cracks are highlighted. The failure mode was bearing failure in the concrete, followed by bolt fracture.

It should be noted that the missing concrete in the centre of the panel is a result of the shear lug action and the four holes in the panel are the locations of the through bolts. Investigations are still ongoing on the effects of shear lugs in bolt-connected panels. Overall, this connection system performed very well under static loading, with peak loads per through bolt exceeding the peak load per shear stud for the control test.

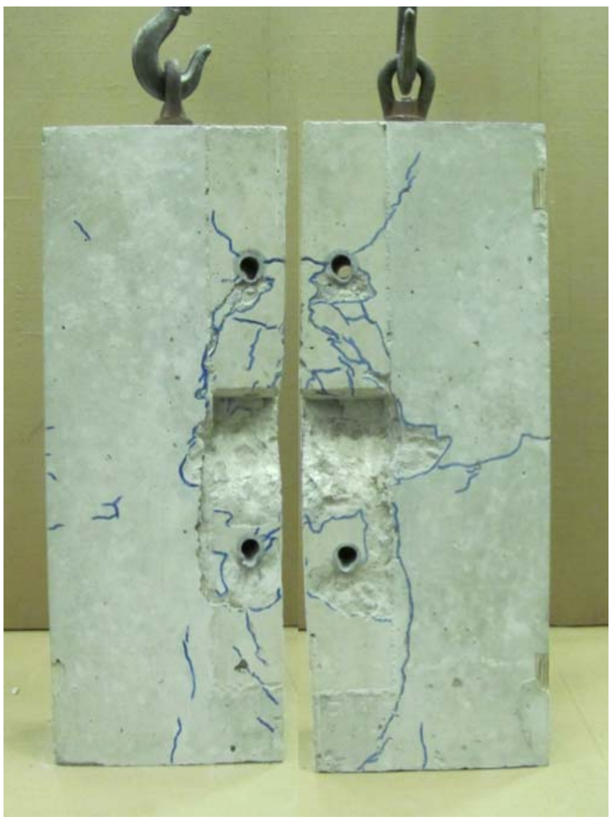

Figure 7: One slab of a failed push test specimen with a shear lug and bolted connection.

The challenges that remain, if this system is to be further developed, are the high construction tolerances needed to make it work practically, as well as testing and analysis of the system under fatigue loading. This second issue may be critical, since the proposed arrangement currently has the shear lug welded to the girder flange.

Summary and Research Needs

All of the shear connection systems that have been investigated at UW to date have an important difference with respect to the other solutions that are currently being used or researched, i.e. that the shear connection can be made mechanically, without the need for grouting. This attribute is important for ensuring that the connection can be made rapidly. It also offers advantages for temporary or portable structures, such as access road bridges for the resource sector or emergency structures. In addition, this attribute may be beneficial when the alternatives are compared on the basis of life-cycle cost, with the possibility of easy and rapid deconstruction at the end of the service life taken into consideration.

The through-bolt connection system has been shown in push-out tests to exhibit high initial stiffness and ductility, but a relatively low initial slip load. Further research is needed to find ways of increasing this slip load and/or to confirm that economical structures can still result, even if this low slip load is considered in the design. In addition, girder tests and fatigue tests are needed to confirm the effect of this slip on girder behavior and performance under cyclic loading.

The panel end connection system needs to be validated in girder tests under static and cyclic loading. With this system and with the use of match cast shear lugs, fatigue may be critical, since both require that a plate or plates be welded to the girder top flange and heavily loaded.

Lastly, for any of these connection systems to make sense economically, they must be used in conjunction with similar “grout free” panel-to-panel connections. Some research on this subject has already been carried out by others. Significant challenges remain though in the development of constructible, effective, and durable “grout free” panel-to-panel” connections.

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