Analysis of Headless Shear Stud Connections

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ABSTRACT

Highway bridges are exposed to numerous elemental and loading issues that are extremely difficult for a designer to anticipate and account for during design. The current state of practice is to design a bridge deck for a certain life span and then turn the bridge over to maintenance personnel who attempt to prolong the life of the deck through a variety of repair and rehabilitation measures. These repair measures are rarely, if ever, considered during the design process of the bridge deck. Numerous researchers have looked at making bridges, specifically decks, more repairable. The majority of these research efforts have focused on the bridge deck system as a whole. Other researchers have looked at individual elements of the bridge deck to girder connection to see if the required strength could be achieved while making the connections easier to take apart. One of the main components in the bridge deck to girder system is the steel shear stud connection, which is used to create composite action between the deck and the girder. Numerous researchers have studied this connection from a strength perspective, and the strength equations for the shear connection have been codified. Shear connections using headless studs have been researched as well, but always as a part of a larger deck to girder connection system. The headless stud has never been researched to see how it responds to a shear loading. This study looks at headless studs with varying levels of debonding along the stud shaft to analyze the impact on the load resistance that the levels of debonding would have. Granular materials for the shear transfer of load are also looked at. The results show that, as expected, the headless, debonded shear studs can carry less load than a bonded stud, but the difference in load carrying capacity is within the suggested over-estimation range of the codes that other researchers have suggested. These results suggest that the use of headless, debonded shear studs in a deck to girder connection is a feasible way to make that connection more repairable.

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1. Introduction

1.1. The Problem with Bridges Being Exposed to the Environment

Highway bridges are structures intended to span 20-40 feet, just enough to clear two lanes of traffic. The most economical form of a short span highway bridge has been found to be the girder bridge. These bridges are built and maintained by state or local governments, who are concerned about the initial cost of these projects. In recent years, the American Society for Civil Engineers (ASCE) has given the country's bridge inventory an overall grade of C (ASCE Report Card). Since these bridges are continuously exposed to the environment and varying loading patterns over their lifetimes, it has been very difficult for the agencies responsible for the bridges to keep them maintained. While this maintenance is a multifaceted issue with economics, politics, environment and engineering involved in the solution, the fact remains that the bridges must be repaired.

One of the major challenges of keeping a bridge inventory maintained is that the rate of deterioration of the bridges is anything but constant. A few examples of how agencies attempt to predict when a bridge will need maintenance are the following:

- Tserng attempted to break the bridge itself into the different segments that need repair. He proposed a total of 20 different portions of the bridge that need to be addressed accurately for a complete bridge to be designed and maintained. (Tserng, 2007)

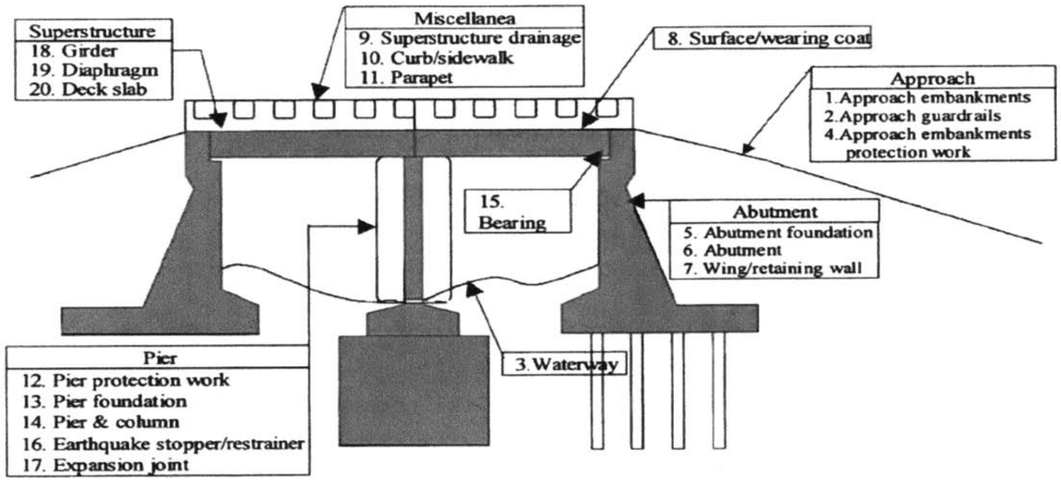


Fig. 1. Locations of 20 predefined elements of a bridge

Figure 1: 20 Elements of a Bridge, All of Which Need to be Connected Together and are Subject to Deterioration (Tserng, 2007)

- Agrawal, et al. established a rating criteria for New York State bridge inspectors to rate the condition of deterioration on New York State (NYS) bridges. With over 17,000 highway bridges in the NYS inventory, their rating scheme had to be very robust. Their study used Weibull distributions to determine how the observed state of a bridge would affect the bridge's overall condition rating (CR). With the CR of 1 to 7, NYS then decides on budgeting and executing maintenance on their bridge inventory. (Agrawal, 2010)

Rating	Description
7	New condition; no deterioration
6	Used to shade between ratings of 5 and 7
5	Minor deterioration but functioning as originally designed
4	Used to shade between ratings of 3 and 5
3	Serious deterioration or not functioning as originally designed
2	Used to shade between ratings of 1 and 3
1	Totally deteriorated or in failed condition

Figure 2: Agrawal's Rating System developed to Rate the Conditions of the Bridge Elements in NY State (Agrawal,2010)

- Oh, et al. did extensive research to create a predictive model that would take into account a bridges current state, its environment, and expected loadings to predict when maintenance would need to be done on the deck. (Oh, 2007) The study, based in Korea, looked at the wheel loading and environmental factors that would deteriorate a bridge deck's capacity. The attached flow chart (Figure 3) shows just the input information required to determine the maximum load effects on a bridge deck.

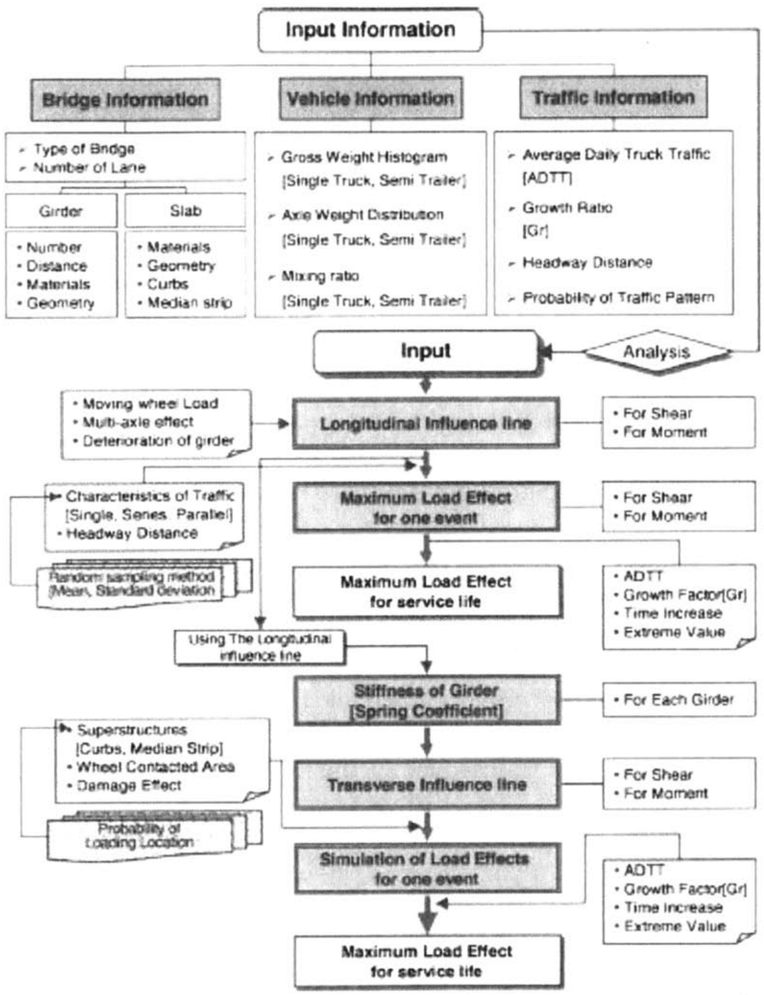


Figure 3: Oh's Maintenance Prediction Flow Chart for Bridge Owners (Oh, 2007)

Their study also created deterioration models that predict how the bridge deck will deteriorate over time based on chloride penetration, corrosion initiation, area loss of rebar and fatigue strength of the deck under moving wheel loads. Using probabilistic estimates, their model predicts that a deck will fail in *t* years given that it has survived for *T* years.

- Kim, et al. used combined multiple regression and GIS technology to identify the critical sources of deterioration of the bridges in North Dakota. Their research found that the age and the structure of the bridge, and traffic volumes were the most significant factors affecting the serviceability of the bridges sampled. (Kim, 2010)

The key aspect of these studies is that they attempt to predict how maintenance personnel should react to a deteriorating bridge inventory. Bridges will always be exposed to varying loadings and the environment. It is a reasonable assumption that they will also be exposed to an ever increasing loading from larger trucks that run more frequently. A bridge would not have been built if each of the two sides of the gap did not need to travel to the other side. With such widely varying factors that lead to deterioration, it is next to impossible to take the current bridge inventory and predict when something will fail. This inability begs the question, is there a better way to keep our bridges functioning in the future?

The bridge industry may do well to look at the industry that uses their infrastructure, namely, the automobile industry, for ways in which to keep their product functioning for longer. An automobile is made up of many removable parts that can easily fail. Consequently, the future maintenance of an automobile is taken into account during its design. If bearings are expected to fail, then those bearings are made to be accessible and able to be replaced. The concept of replaceable parts made automobiles capable of being utilized by the general public. It may be time to take this same concept and apply it to the roads on which automobiles ride.

The goal of this thesis is to look at a subset of the many interrelated parts that need to be studied in order for a bridge to be as repairable as an automobile. This thesis shows that the complex connections of a bridge can be studied more closely. Using this data, a bridge can be designed like an automobile with future repairability as a primary goal of the design process. While a more complex bridge design will increase the initial cost, the life cycle cost is expected to be reduced by the lower maintenance costs.

1.2 Four approaches to combating the problem of bridge exposure

In order to keep the bridge inventory functional, researchers and practitioners throughout the country and the world are exploring several different avenues. These efforts combined with existing technologies constitute the state of the art for bridge construction and maintenance.

In order to guide the efforts of the bridge industry, the American Association of State Highway and Transportation Officials (AASHTO) published the Grand Challenges currently facing transportation officials. These challenges are as follows: 1. Optimizing Structural Systems 2. Accelerating Bridge Construction 3. Advancing AASHTO Specifications 4. Monitoring Bridge Condition 5. Contributing to National Policy 6. Managing Knowledge. (AASHTO, 2005) Figure 4 contains the details of Grand Challenge number two, Accelerating Bridge Construction.

• Implementation and further development of **rapidly assembled** connection details and joints that are constructible, durable and **repairable**
• Development of **maintenance needs**, accessibility, **repairability**, and inspection criteria
• Implementation and further development of design considerations for hardening of existing structures and **rapid recovery after disasters** (natural and manmade)
• Development of prefabricated seismically resistant systems, including substructures
• Development of more efficient modular sections
• Implementation and further development of cost analysis and risk assessment
• Development of quality assurance measures for accelerated techniques for superstructure and substructure construction
• Implementation of advanced materials and continuation of Materials research, e.g., high performance materials, materials durability,
• Implementation of and further development of contracting strategies that encourage speed and quality
• Identification of technical and cultural barriers, Database of Successes w/ costs

Figure 4: AASHTO Grand Challenge #2 (emphasis added) (AASHTO,2005)

Within the first three bullets, AASHTO mentions repairs or repairability four times. The agency is obviously encouraging the bridge industry to make their product more repairable. In order to meet AASHTO's Grand Challenge of making bridges more repairable, the industry and academia have taken three different approaches.

1.2.1 Bridges are being made stronger through different forms and materials.

Researchers at the University of Michigan have been addressing repairability through protection. Historically, one of the points on a highway bridge that deteriorates is the joint between the bridge itself and the approach ramps that come up to the bridge. This joint, usually very near the girder seat, allows water to penetrate to the underside of the bridge and reach the girders. When added with de-icing salts in the winter time, this water penetration causes corrosion and weakening of the girder at the point that it is transferring load to the abutment. Under current designs, this corrosion eventually leads to the replacement of the entire girder, despite the fact that only the end of the girder is excessively corroded.

- Yang, et al. have conducted research creating a self healing cement. When microcracks form in the cement, the water that enters those cracks hydrates the cement to close the cracks again. This material has shown immense ductility and is being field tested as a replacement for the joints and sealants currently used. When successfully applied, the self healing cement removes the access point for water penetration that causes the debilitating corrosion. (Yang, 2007)

-Li, et al. developed and tested an ultra ductile engineered cementious composite (ECC). Their fiber reinforced material was able to withstand much larger amounts of deflection than regular concrete materials. Li found that the ECC displayed strain-hardening properties similar to steel. The ductile strength and strain-hardening aspects of the material are suggested to make the material more applicable to retrofit and repair of civil structures than normal grouts. It should be noted that the tensile and compressive strengths of the ECC were very similar to the Fiber Reinforced Composites (FRC) and mortars that it was compared to during testing. Despite these similarities the overall structural strength and ductility when ECC was used, made the overall system better. (Li, 2000)

- Klaiber et al. studied the applicability of using a form of strengthening commonly seen in Europe for its applicability on American bridges. This form of strengthening is the application of External Pre-stressing. This concept applies cable strands to the exterior of the bridge in order to increase the flexural strength of the girder bridge. Klaiber, et al looked at the use of post-tensioning on simple span and continuous span bridges. They concluded that the application of post-tensioning was a feasible and viable technique for increasing the flexural strength of a bridge. (Klaiber, 1990)

The motivation for Klaiber, et al. was that the traffic volume as well as the vehicle size had greatly increased since the majority of the bridges in the United States were built in the 1940s and 1950s. The need to increase the capacity of the bridges to reduce fatigue and overstress deterioration drove them to look at what could be added to a bridge to increase its capacity.

Klaiber first studied the simple span and continuous span bridges in the laboratory. With satisfactory results from the lab tests, he then instrumented and post-tensioned one simple and one continuous span bridge. Both of the bridges studied in the field showed promising results after having external post-tensioning added to them. He found that the addition of post-tensioning reduced the dead load strains on the bridges allowing them to carry more live load. It should be noted that his system did not seem to reduce live load deflections or influence live load distributions. Klaiber concluded that "post-tensioning is a viable, economical technique for flexurally strengthening simple span and continuous span steel-beam and concrete-deck composite bridges." (Klaiber, 1990)

- McRae and Ramey published a set of two papers emphasizing the importance of meeting both theoretical as well as construction and cost considerations when looking at the rehabilitation of a bridge. The focus of their study, funded by the Alabama State Department of Transportation, was to see how adding additional girder lines to a bridge would affect the bridge's structural response. Their concern was the deterioration of the bridge decks throughout Alabama. By adding additional girder lines, they proposed that the deck would deteriorate less rapidly under applied loadings.

Their theoretical approach focused on adding additional girder lines between existing girders and that action's impact on the deck, existing girders, bent caps, foundations and abutments. They approached the study from the following three perspectives: 1. Strength of Materials 2. Structural Analysis 3. Code Assessment. From their theoretical analysis, they found that the deck strength would be approximately doubled, and deflection (which induces cracking) would be significantly reduced. These affects stemmed from the decreased girder spacing and subsequent stiffening of the bridge superstructure. The remaining fatigue life of the existing girders would also be increased by about a factor of eight. Their concerns about the additional girder lines on the existing substructure found that the bridges should operate within an acceptable factor of safety. (McRae, 2003, PART 1)

The other consideration of McRae and Ramey was the practical considerations of adding additional girder lines, which included the cost and constructability of adding girder lines. McRae and Ramey concluded that the cost would be quite high since the additional girder line would require extensive modification of the bridge structure, specifically the bent caps. The additional girder line would reduce the rate of deck deterioration, but the repair of the deck would be in addition to the cost of adding a girder line. (McRae, 2003, PART 2)

1.2.2 Bridges have been designed to be able to be repaired.

- Newport Transporter Bridge. The Newport Transporter Bridge is located in Newport, England. The bridge spans the River Usk and was built in 1906. (Newport Transporter Bridge) Designed by a French engineer, Ferdinand Arnodin, the bridge uses the transporter bridge concept to ferry passengers and vehicles across the river. The basic concept of the transporter bridge is a highly elevated truss structure with a gondola suspended beneath it. The gondola picks up vehicles, passengers and cargo to ferry them across the river. While this is a very rare (only 8 exist in the world) type of bridge, the design methodology and systematic maintenance approach used by Arnodin is worth examining.

Ferdinand Arnodin (October 9, 1845 – April 24, 1924) was a French engineer who worked with various types of bridge forms. He was known for being a pioneer, if not the inventor, of the transporter bridge concept. He also did numerous projects to refurbish deteriorating suspension bridges. He often would replace the suspension bridge with a combination bridge that contained components of a cable-stayed bridge and a suspension bridge. One of his major focuses was to enhance the repairability of the structures he refurbished and designed.

When he was commissioned along with R.H. Haynes, the borough engineer of Newport, to build the Newport Transporter Bridge, Arnodin used his experience and desire to create a maintainable structure in the design. Throughout design, Arnodin kept detailed notes not only on how the bridge would operate but also on how the different pieces of the bridge could be replaced. He advocated a systematic replacement of parts, to include the suspension cables, in order to distribute the cost of repair over the lifetime of the bridge. Arnodin envisioned a bridge that could remain operational indefinitely through this systematic approach to maintenance. His notes can be essentially interpreted as a maintenance manual for the Newport Transporter Bridge.

For many reasons, Arnodin's manual and replacement parts plan were not implemented on the Newport Bridge. His manual was written in French and was not immediately translated to be useful to the English engineers in charge of the bridge. Also, budget constraints prevented the systematic replacement of parts from being done initially. As with today's bridges, owners do not like replacing parts that have not exceeded their useful life. If Arnodin's replacement system had been used, parts would have been replaced before they failed or possibly even before showing signs of fatigue. Consequently, his manual and proposed maintenance system were not adopted. (Mawson, 2000)

Because of the lack of attention to maintenance on the Newport Transporter Bridge, the structure began exhibiting signs of failure by the 1960s. By 1985, the structure was in such a poor maintenance state that it was closed to use. In 1991, the bridge was so poorly maintained that is was impossible to do any work on it without conducting a major overhaul. Instead of closing the bridge permanently, Newport decided to refurbish and reopen it. By 1995, the bridge was able to be used again. During the restoration, Arnodin's maintenance manual proved to be a great help to the engineers in charge. He had provided a way for every part with a fixed service life to be removed and replaced. However, Arnodin's work on his maintenance manual did not prove to be all inclusive. In the end, nearly half of the budget spent from 1991-1995 on the bridge's repairs was for work that Arnodin had not anticipated. (Mawson, 2007)

Arnodin's attention to the "structural redundancy, replaceability and maintainability" (Mawson, 2007) of the bridge during design proved to be very useful even 90 years after the bridge was constructed. The fact that he did not anticipate all of the maintenance requirements shows that even when a design is focused on repair, the designer's efforts are still limited, because predicting how a structure will fail is immensely difficult.

- I-93 Bridge Replacement project. A modern example of how bridges are repaired is the Medford Bridge Project conducted by the Massachusetts Highway Department (MassDOT) in 2011. With a vehicle usage in the ten's of thousands per day, the I-93 corridor that runs through Medford, Massachusetts, is a major artery into the city of Boston. When the bridges showed major signs of deterioration, such as large potholes appearing in the bridge deck, the Massachusetts Highway's bridge division had to determine how to replace or repair the bridges. A major design constraint was that traffic would be negatively impacted with any long term closure of the roadway.

The short period of time that the state made available for construction drove the project's design. From the appearance of the first pothole in 2010 to the project's completion in September

2011, the MassDOT had to move much more rapidly than they would have under normal business conditions. The result was a set product of "innovations MassDOT used to accelerate the bridge replacements includ[ing]: design-build procurement, a prefabricated bridge elements system, and a special rapid-setting concrete. By replacing the bridges with modular superstructure units that were fabricated off-site, MassDOT eliminated years of work in the roadway." (Fast 14)

Each bridge was removed, replaced and opened for traffic from Friday evening to Monday morning of one weekend. The project showed that with today's technologies the speed with which a highway bridge can be replaced can be greatly accelerated with today's technologies. By essentially replacing the exact same superstructure which had been pre-fabricated offsite, the MassDOT kept the highway open for commuter traffic every weekday during the project's duration. The extreme emphasis on speed led to a remarkable accomplishment in bridge construction engineering.

While the bridge was erected extremely quickly, the future repair of the bridge was left in question. The innovative rapid-setting concrete was not expected to be extremely durable, yet the designers did not account for the need to repair that connection in the future. If that connection or any other on the bridge needs to be repaired, the common practices of bridge engineering will have to be used. When the bridge decks need to be replaced, it is quite possible that a similar project to the Fast 14 will need to be conducted again.

1.2.3 Maintaining a Bridge during its service life.

Current bridge maintenance activities are not within a designer's purview. While making connections accessible or at least able to be inspected has been a focus for many years, the designer is focused on the ability of the bridge to support the required dead and live loads. Maintenance has been left to bridge owners, most often a state or local government, to maintain the bridge once it has been constructed.

The maintenance activities done to increase or extend the useful life of a bridge fall into two categories: rehabilitation and repair. The repair of a bridge means to take a component of the bridge that no longer serves as it should and bring it back to a serviceable level. Repair can be done on the steel and concrete members of a bridge without changing the capacity of the bridge. The rehabilitation of a bridge means to alter the bridge in a way to improve its performance. Some examples of rehabilitation are adding elements to a bridge to combat earthquake loading or foundation scour, or increasing the redundancy of supports to add redundant load paths or reduce fatigue loading on critical members. (Khan,2010)

1.2.4 Another approach to making bridges function longer

While the three previous focus areas all make bridges more durable or able to be replaced quickly, to make bridges more repairable requires a greater understanding of the connections that make the individual pieces work together. The basic premise of Ferdinand Arnodin was when he did not expect a part to last indefinitely he designed it to be able to be removed and replaced. In order to remove a part from today's highway bridges, we must understand the connections that make up the bridges. As it will be shown, these connections have been studied at great length, but the basic assumption behind those studies did not lend itself to future repair.

The basic assumption of bridge design and construction is to create a bridge to meet a certain design life. That design life determines the magnitude of the loads that are designed to be resisted. To resist these loads, the structure and its connections are made continually stronger as the design life increases. This assumption that stronger connections makes a more durable bridge is similar to Arnodin's approach of taking great care in designing and constructing the substructure of the Newport Transporter Bridge. He anticipated that the substructure would not be able to be repaired thus it needed to be extraordinarily durable. (Mawson, 2007) The problem arises when we attempt to make every connection on a bridge very durable. Something will eventually fail, and if everything has been made to last as long as possible, it makes the repair of those connections exceedingly difficult.

To see what could be done when a part of a structure is anticipated to fail, we need only look at other current construction projects. The Apple computer stores around the country are made completely of structural glass. Structural glass, while very strong, is also very brittle and prone to catastrophic failures. To design a building made completely of structural glass, Apple designers and contractors had be 100% confident in the precision of their construction. But even with complete precision in the design and installation, the glass will still fail at highly unpredictable rates. Consequently, Apple also designed the buildings so that each piece of glass could be removed. To remove each piece of glass without affecting the ability of the whole structure to remain intact, an intimate knowledge of the connections between each piece of structural glass was required. These connections had to be accessible, removable and replaceable. All of which had to be accomplished without having to take apart the rest of the structure.

Picture 1: Exterior of Apple Building, Boston, MA

Picture 2: Interior Staircase Connection

Picture 3: Exterior Façade Connection, Boston MA

Picture 4: Interior Staircase Connection 2

It is not realistic to say that the care given to the design and construction of the Apple stores can be applied to the thousands of highway bridges throughout the country. However, it is a fair comparison to say that the unpredictable nature of the deterioration rates within the bridge components is very similar to the unpredictable failure rates of structural glass. The strength of the Apple store buildings is the ability to take them apart piece by piece regardless of which part fails. The same application could be used on highway bridges, but it requires a more extensive knowledge of the connections that join the pieces of a bridge together than is currently available.

1.3 Problem Statement

By applying the same type of understanding of connections as seen in the Apple store application to bridges, especially the connections between the parts of the bridge that deteriorate the most rapidly a designer could make a bridge more repairable. One of the most rapidly deteriorating parts of a bridge is the deck. By focusing on the connection between a deck and its supporting girders, one could make a bridge more repairable. Relaxing the rigidity of the connection between the shear stud and the grout or concrete around it is one way that the connection between the deck and the supporting girders can be made more repairable. The obvious concern is that this connection creates the composite action that allows a bridge to be built lighter than with a non-composite deck. The shear connection should not be removed completely, it should be studied so that the deck is made more removable. Instead of making the bridge deck stronger through using a longer design life, one could make it more repairable by understanding the connections, especially the shear connection, more.

The specific shear connection that will be studied in this thesis is a headless stud shear connection. As it will be shown, a headless stud shear connection has been used as part of a bridge system but never studied on its own. My objective is to test specimens varying levels of bond between the steel stud and the surrounding grout to determine if headless, debonded studs can resist shear loadings.

2. Literature Review

Several different researchers have looked at many of the connections that make up the bridge system. The research efforts have ranged from looking at individual connections to complete bridge systems. A review of the research efforts to date will show that the connections of a bridge can be made to be more repairable while not sacrificing the performance of the bridge.

2.1 Porter

Porter, et al. studied the transverse joint connections on precast deck panels. Porter's emphasis was finding a means of connecting the joints with the previously studied and accepted in design criteria of 200 psi of compression while not using a post-tensioning system. Porter proposed 2 new curved bolt connections, a welded rebar connection and a welded shear stud connection with the intent of being able to replace deck panels individually. Two of his proposed connections were able to carry more moment, on average, than a typical post-tensioned transverse joint. Those two connections, welded rebar and long curved bolt, show that it is possible to have a connection that gives the ability for an individual deck panel to be replaced while still carrying the required forces. (Porter, 2010)

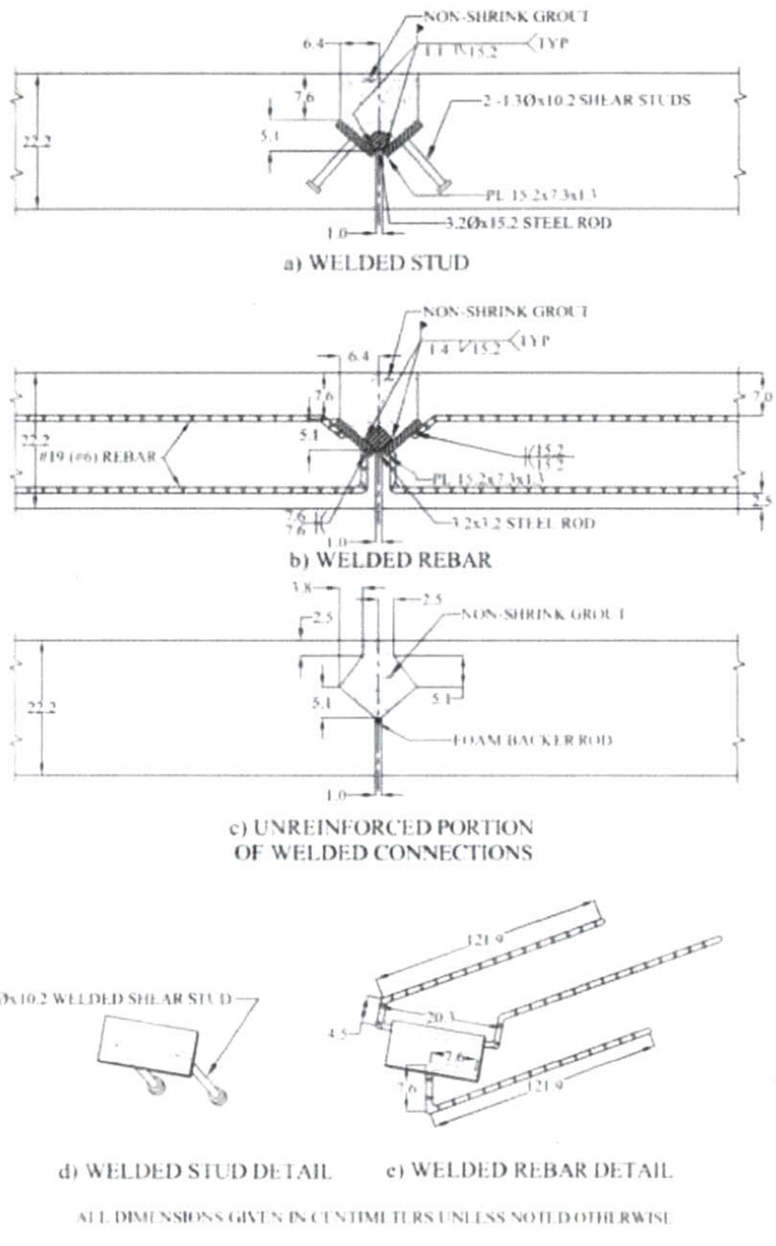


Figure 5: Conventional Connections studied by Porter (Porter 2010)

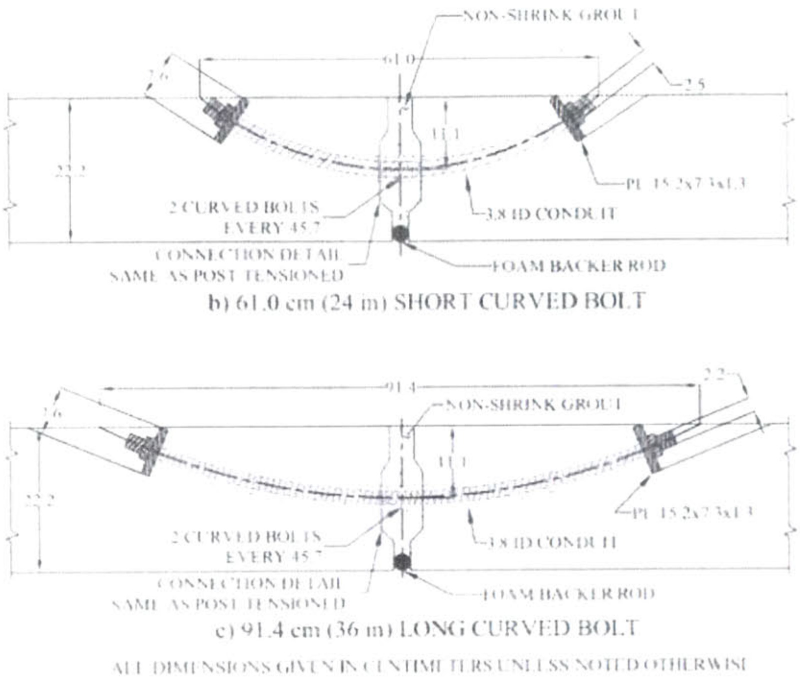


Figure 6: Proposed Curved Bolt Connections (Porter 2010)

As seen in Figure 7, Porter's proposed connections that utilized curved bolts performed quite well in comparison to the conventional connections. His short, curved bolt connection did not exceed the strength of the conventional connections. His long, curved bolt connection performed nearly as well as the post-tensioned connection. The overall average strength of his long, curved bolt connection was higher than the post-tensioned connection, but the post-tensioned connection used a lower strength (5.43 ksi) concrete than the long, curved bolt did (6.07 ksi) for the majority of the tests.

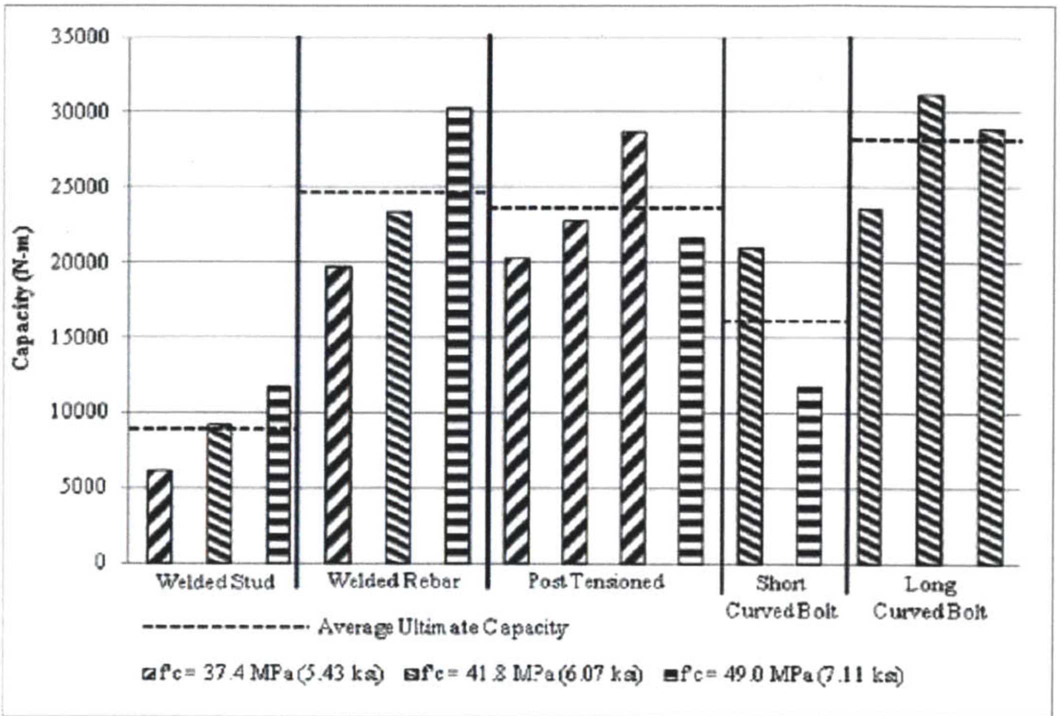


Figure 7: Comparison of Conventional Connections with Proposed Curved Bolt Connections (Porter 2010)

2.2 Tadros

Tadros, et al. studied the application of a more rapidly removable bridge deck system in order to meet the need of limiting the impact to high volumes of traffic that use the country's bridges. The current system of using a roughened, bonded, reinforced interface was seen to have the following drawbacks: 1. Deck removal is difficult and time consuming, especially when trying to preserve the girders that a deck is attached to and preserving the composite (shear) connectors. 2. The top flange of the girder is at risk of being damaged during removal. 3. During replacement, the shear connectors are subject to corrosion. (Tadros, 2002)

With these concerns, Tadros et al. conducted the National Cooperative Highway Research Program (NCHRP) Project 12-41 "Rapid Replacement of Bridge Decks" at the University of Nebraska. Tadros' philosophy was "that bridges should be built similar to parts of a model car that are snapped or bolted together." (Tadros, 2002) The research team developed a full bridge deck system that utilized a debonded shear key interface system and headed shear studs to allow for more rapid replacement of the deck while still achieving the desired composite action.

Tadros' shear key system attempted to provide the same amount of interface shear that had been studied in the past and achieved through a roughened, bonded interface. The key component was the geometry of his system, See Figure 8, which he found could achieve the same amount of interface shear as a flat girder using a bonded interface could achieve. The proposed system consisted of the following: 1. Concrete shear keys formed on the top flange surface of the girder that provide mechanical interlock between the top flange surface and the deck concrete. 2. Sealant applied to the girder top flange surface to break the bond between the surface and the deck placed on top of the flange. 3. U-shaped shear connector bars embedded in the girder web and extended up into the deck slab. (Tadros, 2002)

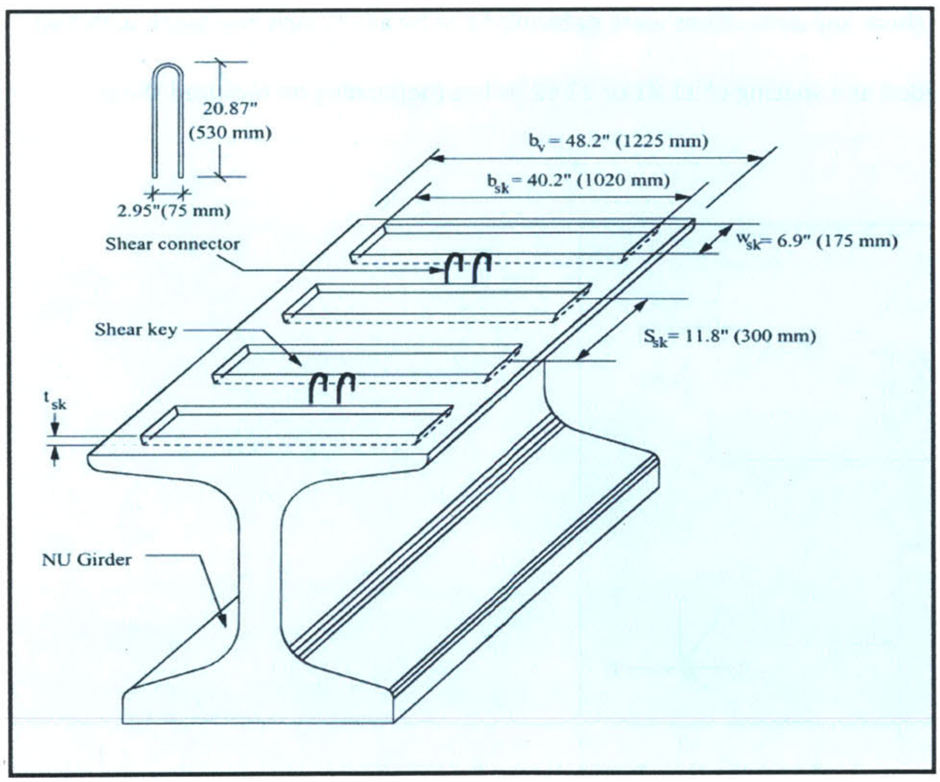


Figure 8: Proposed Shear Key Mechanism (Tadros, 2002)

Tadros designed his shear key mechanism to use the geometry of the interface between the concrete deck and the supporting concrete girder in order to transfer the shear forces between the two. Since the interface between the two is angled, as seen in Figure 9, the two surfaces slide relative to each other and then engage the connecting bars (represented by shear studs in the figure). When the two surfaces slide relative to each other, they cause tensile and shear stresses in the steel connecting bars.

To further describe Tadros' system, as seen in Figure 9, the applied force is **P**. $\mathbf{P_y}$ is the tensile force in the steel connector, $\mathbf{P_{xy}}$ is the shear force in the steel connector, **R** is the bearing force on the side of the shear key and **δR** is the friction force on the side of the shear key (**δ** being the interface friction factor of the two debonded surfaces). He used the bearing on the side of the shear key and the shearing of the base plane of the shear key as two means to design the size of the shear key required. He used these calculations to determine the standardized sizes of shear keys for his complete bridge design. The standardized shear key dimensions were determined to be a 0.75 inch key depth with two No. 5 U-shaped bars provided at a spacing of 11.81 or 23.62 inches (depending on required shear capacity). (Tadros, 2002)

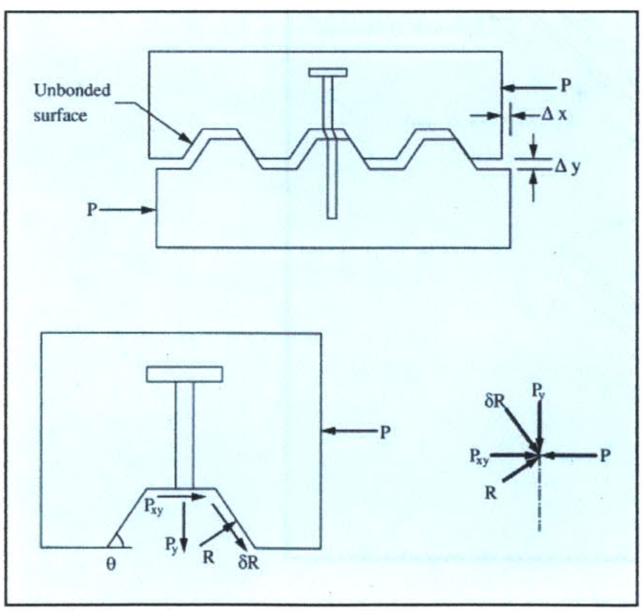


Fig. 4. Shear key mechanism.

Figure 9: Further Description of Mechanism for Shear Key System (Tadros, 2002)

Since the shear forces between decks and girders are transmitted through the shear stud and the removal of grout or concrete around those shear studs is problematic during deck replacement, Tadros looked at the possibility of using a larger diameter shear stud. Being able to use a larger diameter shear stud (typical diameter sizes are three-quarter inch or seven-eighths inch) would allow for fewer shear studs to be used on a typical bridge application. Tadros investigated the possibility of using a 1-1/4 inch diameter shear stud. His research focused on the material the stud would be made from, the method of welding the stud to steel girders with either a modified stud gun or a new welding mechanism, development of a testing mechanism for testing weld quality, and techniques of evaluating structural performance. (NCHRP 407)

As part of the structural performance evaluation in the testing of the large diameter shear studs, Tadros developed a full scale test. Using a 40 foot long W36x160 hot rolled section, Tadros alternated headed and headless studs to anchor a four foot wide by eight inch deep concrete deck. (NCHRP 407) After the application of fatigue and static loads to the specimen, Tadros observed the difference in slippage between the deck and the girder was between 0.00013 to 0.0005 inches. The stress distribution diagrams showed no change in the neutral axis. He observed no major cracks, concrete crushing or stud failure. It must be noted that the focus of this full scale experimental program was the use of the large diameter shear stud. Tadros also conducted timed experiments for different methods of deck removal, but did not differentiate between the time needed to remove concrete from around the headed versus the headless shear studs. (NCHRP 407)

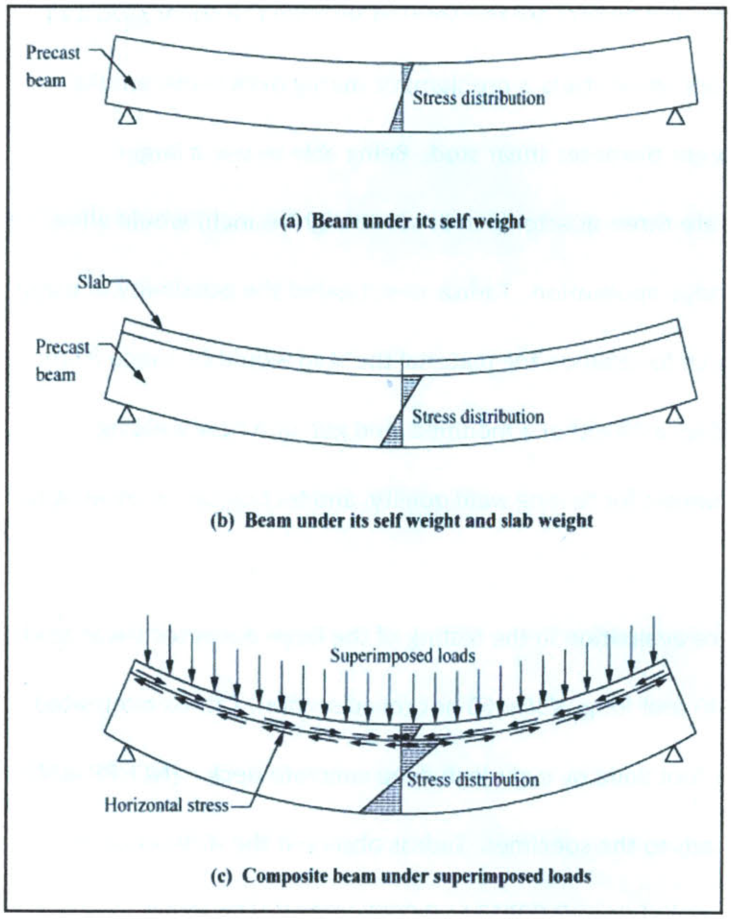


Fig. 5. Composite beam behavior.

Figure 10: Description of Composite Loading (Tadros, 2002)

In his design for the factored horizontal shear stresses, Tadros only used the superimposed loads (see part c of Figure 10). He used this approach for the following reasons, 1. Specifications do not give specific guidance on which loads to use. 2. The girder and slab are in place prior to composite action happening. 3. This design approach had been used by the Illinois Department of Transportation for many years with no reported problems at the time of his experiment. (Tadros, 2002)

Tadros also found that the specifications either did not mention how the factored horizontal shear stress should be calculated or underestimated the actual horizontal shear stresses. For the underestimation, he did not find it to be an issue, since the loads are assumed to be very over estimated.

After extensive analysis and design of a complete bridge deck and girder system, Tadros concluded that the debonded shear key interface system for precast, prestressed concrete girders had several advantages including the following: 1. Since the future removal of the deck was considered during design this consideration would enable a more rapid removal in practice. 2. The shear connectors were better protected against corrosion. 3. The girder's top flange was protected from damage during deck removal. (Tadros, 2002)

2.3 Yamane

Yamane, et al. studied a precast, full-depth, reinforced concrete, panel bridge deck system. The goal of his research was to test a system that would allow for more rapid replacement of a bridge deck system. He noted the following reasons why full depth precast pre-stressed concrete panel systems had not been more readily used included the following: 1. Difficulty of accommodating crowned sloped sections due to the pre-tensioning in the long-line bed. 2. Difficulty in transferring and developing strands in the first maximum negative moment zone located at the external girders. He found that his system addressed these major concerns. (Yamane, 1998)

Yamane's bridge deck system not only consisted of precast, prestressed concrete panels, but he also used a combination of welded headless shear studs in concert with welded threaded studs. To finalize the placement of the deck and create composite action, he utilized grout filled shear keys, leveling bolts and threaded bars for post-tensioning. (See Figure 11)

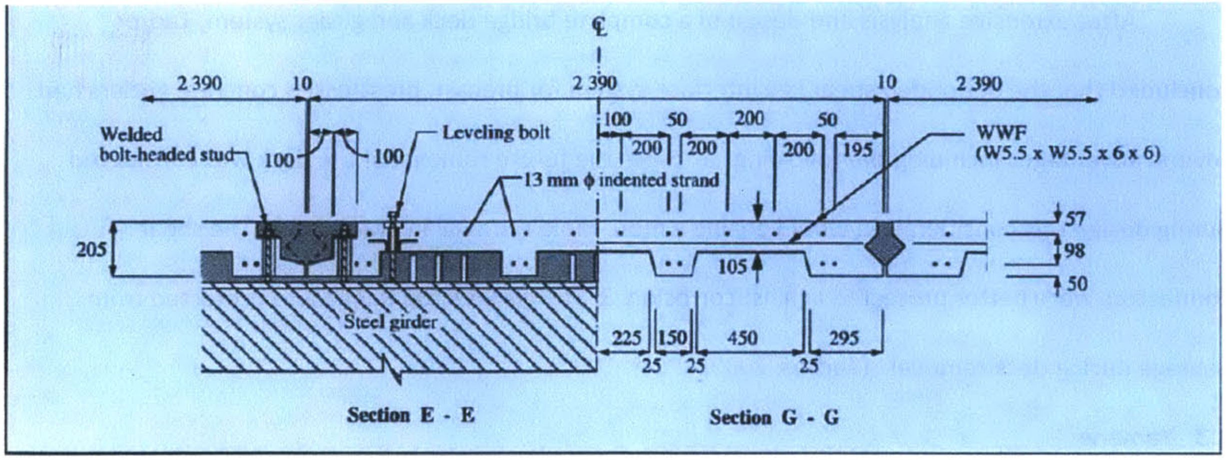


Figure 11: Yamane's Proposed Deck System (Yamane, 1998)

Of note was the use of headless shear studs in Yamane's design. These headless studs were designed to only transfer shear force. Without the heads attached, which are used to combat uplift forces in the deck in traditional bridge design, the headless studs had to be used in conjunction with the threaded headed studs seen in Figure 11. This combination of studs was able to create fully composite action between the attached girder and the bridge deck system while allowing for more rapid removal and thus replacement of the bridge deck in the future. Yamane conducted a finite element analysis of his proposed bridge deck system and found that the stresses in the deck were well within the acceptable range of the AASHTO Specifications.

2.4 Issa

Issa, et al. conducted testing on full- and quarter-scale push-out specimens in order to study the composite behavior of the shear connection in full-depth, precast concrete bridge deck panels placed on steel girders. The purpose of the study was to compare the experimental results with a finite element analysis, as well as the current design code equations. The research program focused on the number of shear pockets in the precast deck panel, the number of studs in each of those shear pockets, and the use of a grouted haunch for connecting the precast panel to the supporting girders. (Issa, 2003)

Of the 28 specimens testing, 14 were full-scale and 14 were quarter-scale. The quarter-scale specimens used one, two, three and four studs per shear pocket. While the full-scale specimens used two, three and four studs per shear pocket. See Figure 12 for the arrangement of the studs in shear pockets. The research program also tested two full-scale specimens with no shear studs to examine the contribution of friction to the connections. (Issa, 2003)

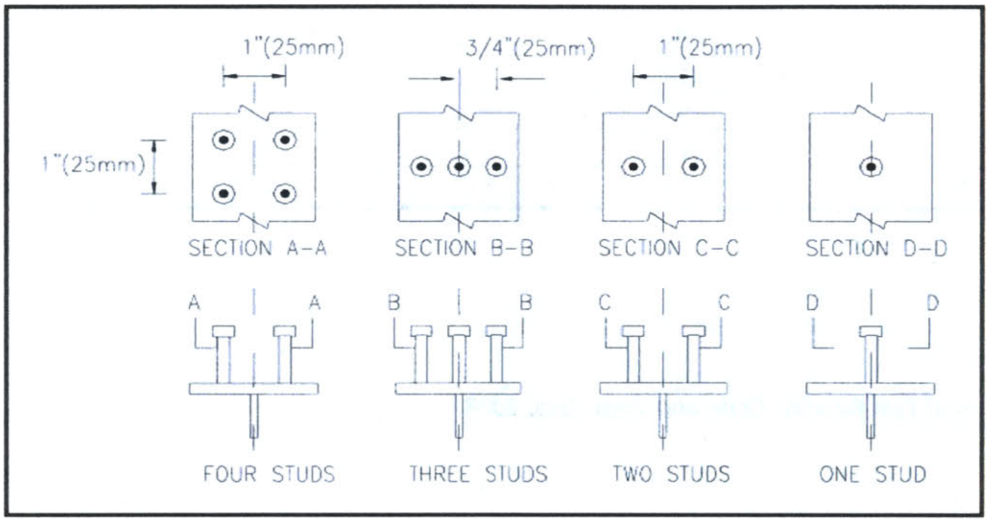


Figure 12: Shear Stud Orientation for Issa's Experimental Setup (Issa, 2003)

All the specimens were loaded with a quasistatic loading with the load, slippage, and ultimate load at failure being recorded. Failure modes as well as crack formation and propagation were noted for all specimens as well. Issa observed that the critical point of slippage ranged from 0.02 to 0.03 inches for all specimens, with the 0.02 inches of slippage being the dominant measurement. The critical point was when the shear studs fully engaged to create the composite action desired in the composite specimens. (Issa, 2003)

The results of this experimental program were compared to the AASHTO specifications and other researchers, namely Viest who pioneered welded stud research in the early 1950s. Viest's research found that the height of the stud had little impact on the strength of the shear connection. See Figure 13 for the comparison of this research program with the AASHTO specifications and Viest's research. The researchers found that the AASHTO specifications overestimated the strength of the shear stud connection by up to 22%. (Issa, 2003)

Table 7. Comparison of test results for shear strength between Viest equations and AASHTO specifications.

Specimen designation	Average observed ultimate load (kips)	AASHTO Standard. Spec,* Article 10.38.5, Eq. 10-67, S_u, AASHTO LRFD Spec.,† Article 6.10.7.4.4c-1, Eq. 6.10.7.4.4c-1, Q_n (kips)	AASHTO‡ LRFD Article 5.8.4.1 Eq. 5.8.4.1-1, V_{nh}, based on $c = 25$ psi, $\mu = 0.7$, (kips)	Viest§ (kips)
FS1P0S	8.5	—	6.9	—
FS1P2S	69.2	72	46	41
FS1P3S	92.9	108	66.24	61.5
FS1P4S	121.4	144	86	82
FS2P0S	15.1	—	13.6	—
FS2P2S	118.9	144	92.7	82
FS2P3S	176.8	216	132.3	123

* $S_u = 0.4d_s^2\sqrt{f'_c E_c} \leq 60{,}000A_{sc}$

† $Q_n = 0.5A_{sc}\sqrt{f'_c E_c} \leq A_{sc}F_u$

‡ $V_{nh} = cA_{cv} + \mu(A_{vf}f_y + P_c)$

§ $Q_{cr} = 5.25d^2 f'_c\sqrt{4000 / f'_c}$

Figure 13: Comparison of Test Results, Code and Viest (Issa, 2003)

2.5 Badie

Badie et al. studied the possibility of extending the AASHTO LRFD limitation on the spacing between shear connectors on steel girders. The current standard limits the spacing to 24 inches. (AASHTO 2007) Badie looked at extending that spacing to 48 inches in order to: 1. Simplify and speed up the fabrication process of the panels. 2. Reduce the volume of the grout filling shear pockets which inherently would reduce the cost of the deck and increase construction speed. 3. Reduce the possibility of water leakage between shear pockets and grout. 4. Allow the designers more flexibility in designing reinforcement by allowing for fewer shear pockets. (Badie, 2010)

Badie focused on four main concerns that should be taken into account whenever dealing with the application of shear studs to a concrete decking system. These four issues are as follows: 1. Longitudional splitting cracking in the deck slab over the girder lines. 2. Separation between deck slab and girder. 3. Bearing failure of the slab in front of stud cluster. 4. Crushing failure of grout at the stud base. (Badie, 2010) Badie related the first three issues to the spacing of stud clusters only and the fourth to the size of the stud only. These four issues are integral to understanding the interaction of the stud with the concrete deck of any bridge system.

As seen in Figure 14, the attempted horizontal movement of a girder relative to the slab induces a force, **P**, that will both attempt to lift the deck from the girder, $\mathbf{P_v}$, and split the concrete behind the stud with relation to the direction of force, $\mathbf{P_l}$. Badie reviewed the work of others that ignored the vertical dispersion of the force, $\mathbf{P_v}$, which resulted in the whole force, **P**, being used as the capacity of the stud. The same behavior was expected to happen with a single stud or with a cluster of studs. (Badie, 2010)

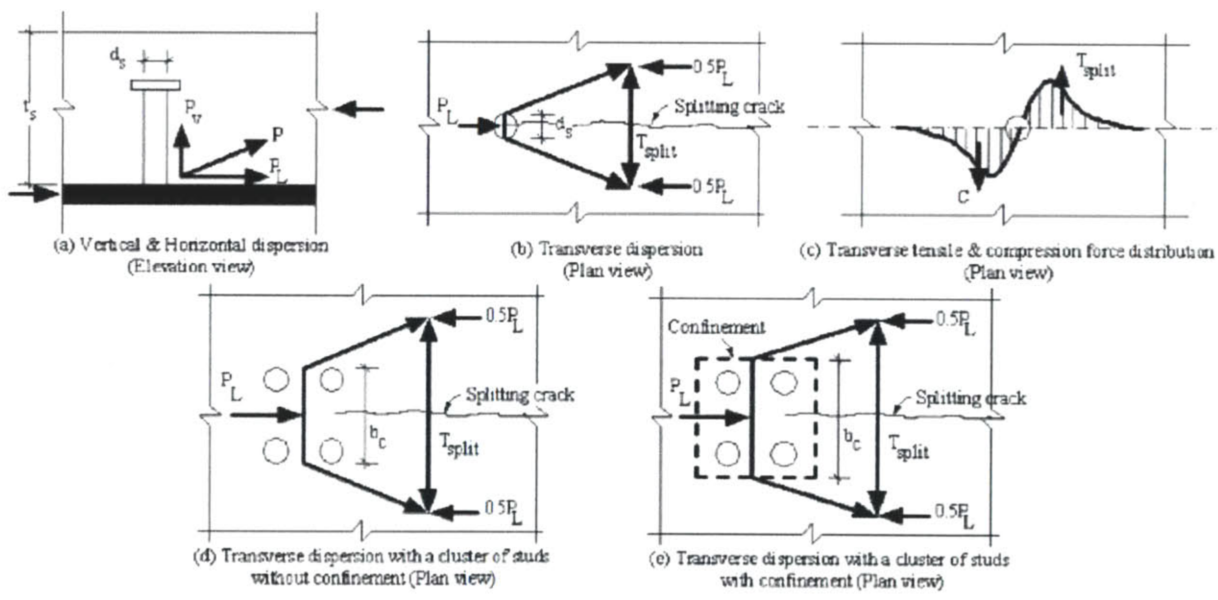


Fig. 1. Vertical and horizontal dispersion of a stud force

Figure 14: Breaking the Force on a Shear Stud and Cluster of Shear Studs into its Components (Badie, 2010)

Badie found minimal research about the separation between the girder and slab. He concluded that the research had shown that full composite action would still be achieved even if a separation between the girder and slab was observed. It is important to note here that the research has all been done with headed shear studs. Badie also noted that the self weight of a deck slab on an actual bridge would assist in holding the deck and slab together.

From other research by Hawkins et al., Badie obtained an empirical formula for determining the bearing strength of the concrete in front of the stud. This is of special concern with large stud spacings, since the failure of the concrete in front of the stud could cause the entire system to fail without fully transferring stresses to the shear stud.

In all shear stud centric composite systems, the stress concentrations at the base of the shear stud are extremely high. Since Badie was using one and one-quarter inch diameter shear studs, he was especially concerned that the concentration of stresses would cause the grout to fail resulting in the total flexural strength of the system being underestimated. He investigated two different types of lateral confinement around the stud clusters in order to protect the grout from this crushing effect.

Badie used both a closed-tie and a steel tube confinement approach for a 4-stud cluster and an 8-stud cluster. (See Figures 15-18) The 4-stud clusters were meant to represent a spacing of 24 inches between shear pockets. The 8-stud clusters represented a spacing of 48 inches between shear pockets in a precast panel system. Badie based this number of studs and spacing on an empirical study he performed in 2002, which showed that one shear stud of one and one-quarter inch diameter needed to be placed every 6 inches for full composite action. (Badie, 2010)

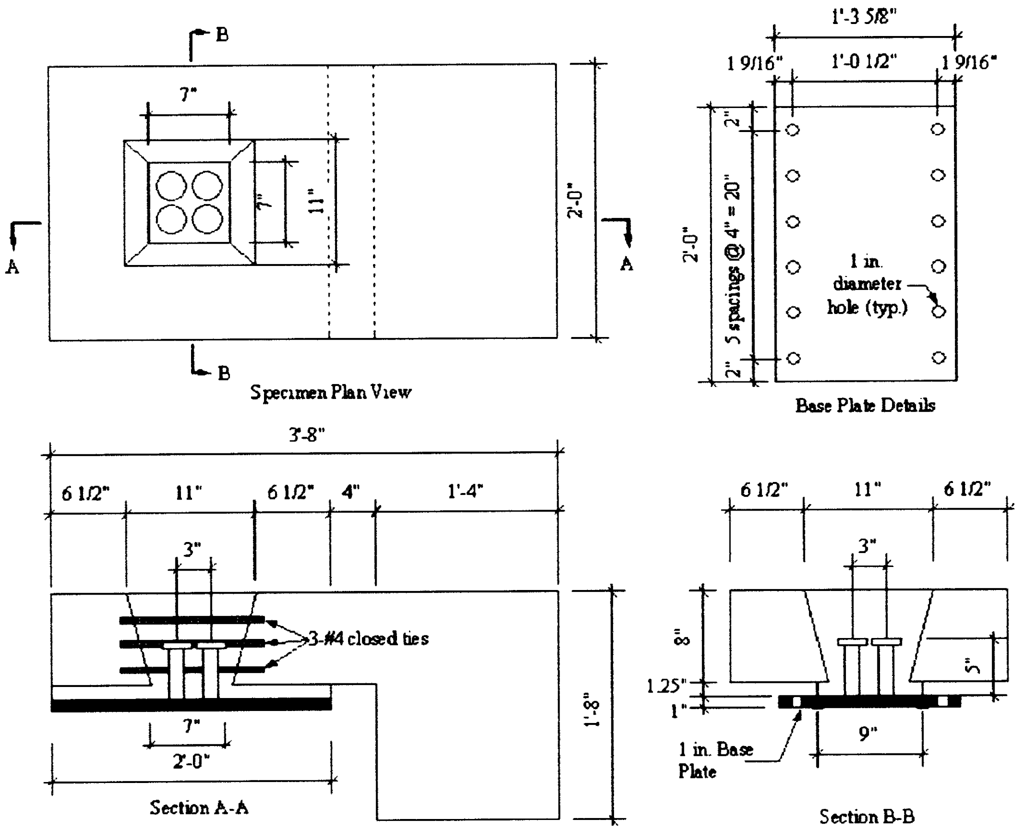


Fig. 2. Concrete dimensions of the four-stud closed-tie confinement specimen

Figure 15: Closed Tie Confinement, 4 Studs (Badie 2010)

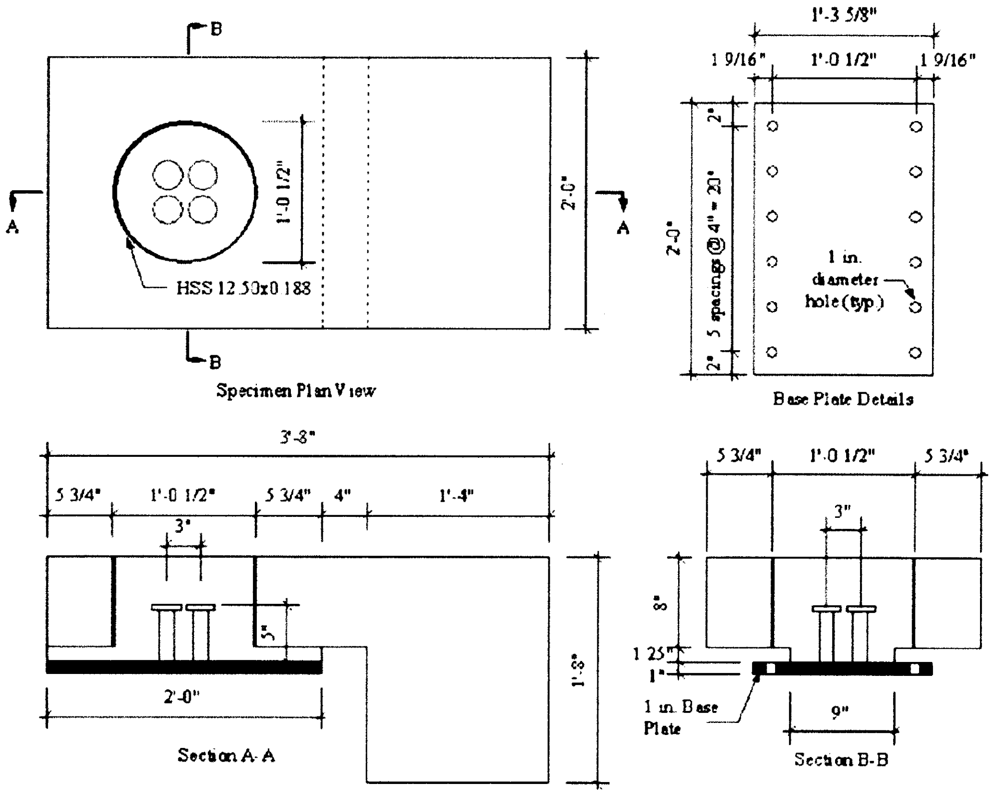


Fig. 3. Concrete dimensions of the four-stud steel-tube confinement specimen

Figure 16: Steel Tube Confinement, 4 Studs (Badie, 2010)

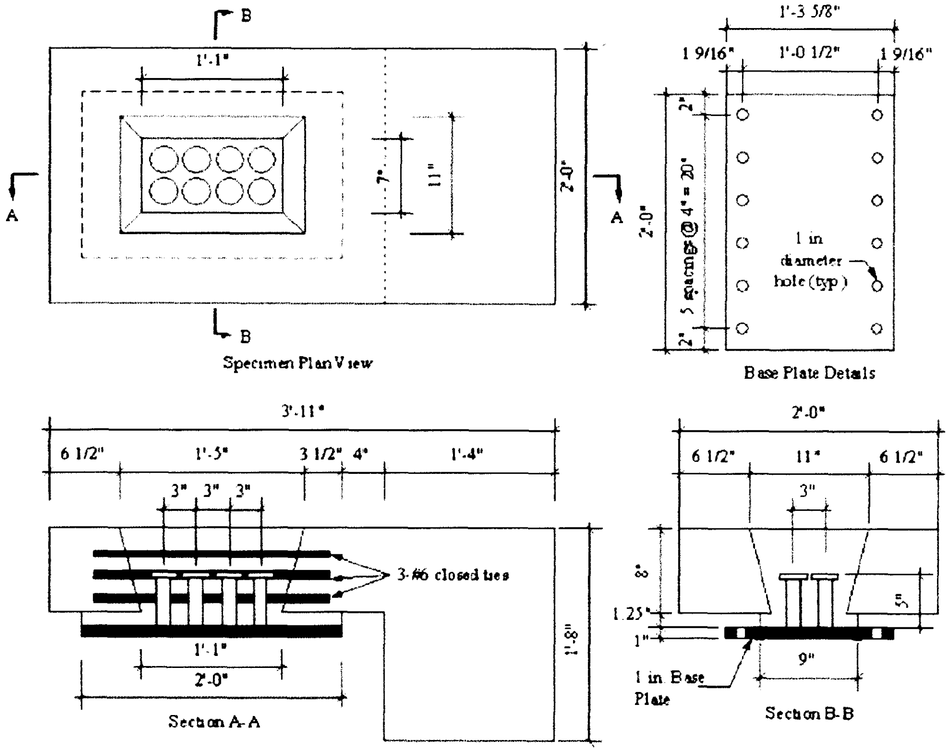


Fig. 4. Concrete dimensions of the eight-stud closed-tie confinement specimen

Figure 17: Closed Tie Confinement, 8 Studs (Badie, 2010)

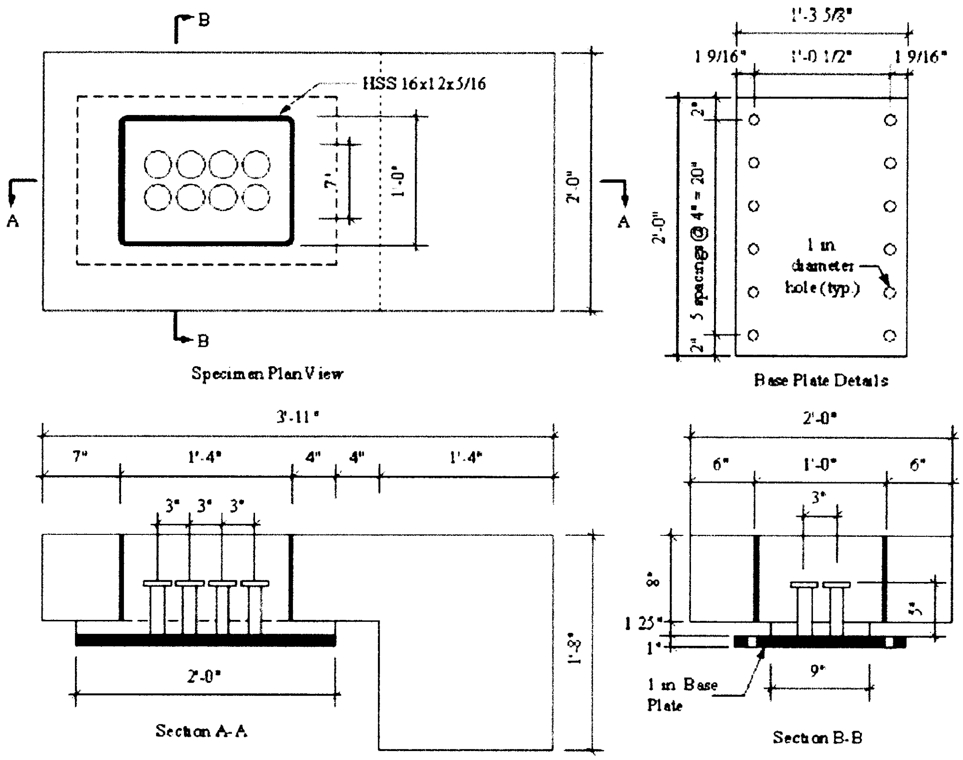


Fig. 5. Concrete dimensions of the eight-stud, steel-tube confinement specimen

Figure 18: Steel Tube Confinement, 8 Studs (Badie, 2010)

Using the apparatus seen in Figure 19, Badie applied a shear load to the specimens in two groups. During the horizontal load application to Group 1, it was observed that extensive cracking occurred in the concrete outside the shear pocket. Consequently, the second group had a additional set of side plates attached. These side plates were expected to replicate the fact that an actual bridge deck would extend to either side of the tested specimen and provide similar confinement. Badie's first group was only exposed to a static load applied at a rate of five kips per second. The second group was statically loaded to fatigue capacity, then the specimens were loaded with two million cycles of the fatigue load. Finally, the second group was loaded until failure.

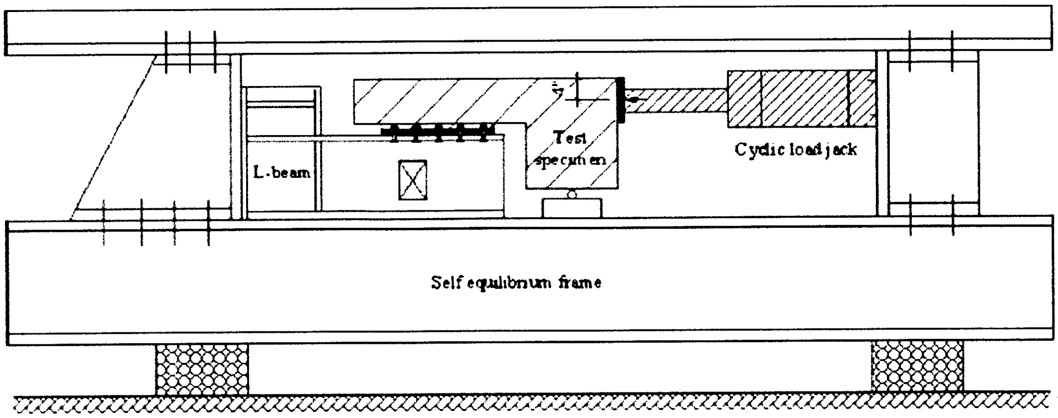


Fig. 7. Test setup of the push-off specimens

Figure 19: Test Setup for Badie Experimental Program (Badie, 2010)

Badie observed the following five modes of failure: slab failure, bearing failure, shear stud failure, grout crushing failure and bond failure (between the lower layer of closed-ties and the concrete). From these observations, the experimental program came to several conclusions which included the following:

1. Fatigue capacity of clustered studs:
    a. No signs of concrete/grout crushing, weld failure or local distress around or inside the shear pockets were observed when the push-off specimens with four-stud and eight-stud clusters of one and one-quarter inch diameter were exposed to two million cycles of fatigue load.
    b. Almost no change occurred in the load-displacement relationship of the push-off specimens after applying 2 million cycles.
    c. Specimens confined with closed tie showed higher displacement compared to the specimens confined with a steel tube. This trend was more pronounced in the specimens made with eight studs.
2. Ultimate capacity of clustered studs:
    a. The test results of the four-stud and eight-stud specimens show that , regardless of the type of confinement used around the stud group, the ultimate capacity did not proportionally increase when the number of studs was doubled.
    b. Regardless of the number of studs, the ultimate capacity of a stud group confined with the steel tube is higher by about 5-15% than the ultimate capacity of the same stud group confined with individual closed tie. The difference is more pronounced with the four-stud group than the eight-stud group.
    c. Regardless of the number of studs per group, the type of stud confinement, and test type (ultimate capacity versus fatigue and ultimate capacity), Eq. 6.10.10.4.3-1 of the *AASHTO LRFD Specifications* (AASHTO 2007) overestimates the ultimate capacity by as much as 60%.
    d. For push-off specimens tested directly for ultimate capacity, Eq. 5.8.4.1-1 of the *AASHTO LRFD Specifications* (AASHTO 2007) and the equation developed by Viest

(1956) correlate very well with the test results. This observation is consistent with the findings by Issa et al. (Issa, 2003)

e. A bond failure between the lower tie and the concrete slab was observed in most of the specimens made with closed individual ties and subjected to the two million cycles of fatigue load. This failure most likely occurred because of the large size of the bar, which led to high stress concentration. Using No. 4 or 5 closed tie may help to avoid this failure.

3. Shape of the push-off specimens:
    a. For future investigation, it is recommended to use symmetric push-off specimens instead of the L-shaped specimen used in this research. However, due to the expected high load that is required to break the symmetric specimen, half- or quarter-scale specimens should be considered.
    b. The failure modes of Groups #1 and #2 show that the side external confinement of the specimen is very important to overcome the limited-width problem of the push-off specimens. All the specimens of Group #1 had slab failure, while almost all of the specimens of Group #2 had stud failure. Unfortunately, no mathematical models are available at the current time to quantify the amount of the side confinement needed to simulate an actual bridge. (Badie, 2010)

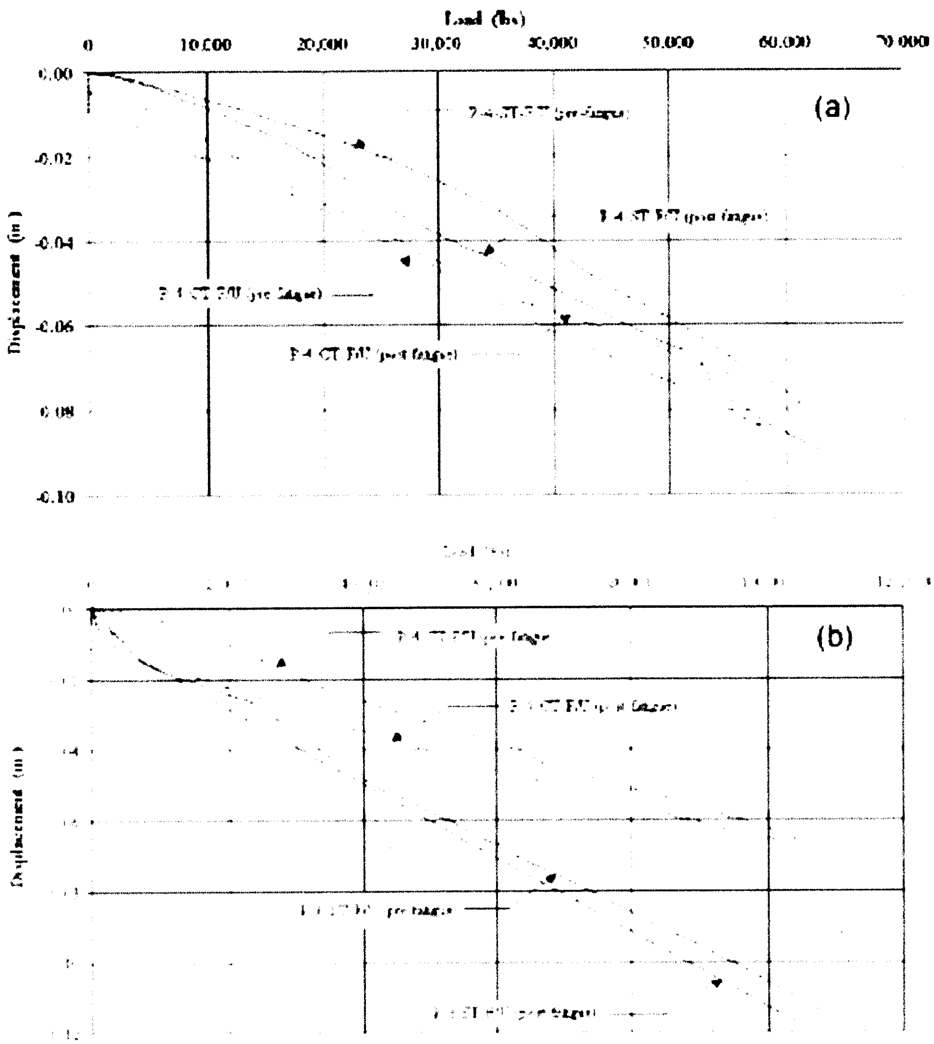


Fig. 8. Load-displacement relationship before and after applying the fatigue load: (a) four-stud specimens; (b) eight-stud specimens

Figure 20: Results of Fatigue Load Experimental Program (Badie 2010)

2.6 Menkulasi

Menkulasi et al. studied the horizontal shear strength of the connection between a precast deck panel and concrete girder. The parameters that he was most concerned with were the haunch height, grout type, applicability of using alternate shear connectors and the area of reinforcing steel crossing the shear connection interface

As shown in Figure 21, Menkulasi noted that the horizontal shear strength of a connection can be approached from the following three sources: ACI 318-02, AASHTO LRFD Specifications and AASHTO Standard Specifications. Menkulasi simplied these three different approaches to one equation $V = C + \mu A_{vh} f_y$ with C=cohesion (also encompassing dowel and other contributing factors), μ=friction coefficient, A_{vh}=area of reinforcing steel crossing the shear plane, f_y=yield strength of reinforcing steel. . (Menkulasi, 2005)

Table 1. Equations for horizontal shear strength.

Code or specification	Horizontal shear strength
ACI 318-02	$V_u \leq \phi V_{nh}$ No ties, clean, roughened surface: $V_{nh} = 80 b_v d$ (lbs) Minimum ties, clean, smooth surface: $V_{nh} = 80 b_v d$ (lbs) Ties provided, clean, roughened surface: $V_{nh} = [260 + 0.6 A_{vh} f_y/(b_v s)] b_v d$ not greater than $500 b_v d$ (lbs) If $V_u > \varphi 500 b_v d$, design according to shear friction section Minimum ties: $A_{vh} = 0.75 \sqrt{f_c'} \frac{b_v s}{f_y}$ not less than $50 b_v s/f_y$
AASHTO Standard Specifications	$V_u \leq \phi V_{nh}$ No ties, clean, roughened surface: $V_{nh} = 80 b_v d$ (lbs) Minimum ties, clean, roughened surface: $V_{nh} = 350 b_v d$ (lbs) Ties provided exceeding minimum, clean, roughened surface: $V_{nh} = 330 b_v d + 0.4 A_{vh} f_y d/s$ (lbs) Minimum ties: $A_{vh} = 50 b_v s/f_y$
AASHTO LRFD Specifications	$v_{uh} A_{cv} < \phi V_n$ $v_{uh} = V_u/b_v d_v$ $V_n = c A_{cv} + \mu (A_{vh} f_y + P_c)$ (lbs) where c = cohesion factor = 100 psi for clean and roughened surface μ = friction factor = 1.0 for clean and roughened surface P_c = permanent compressive force across interface

Figure 21: Design Equation Comparison (Menkulasi, 2005)

Menkulasi expected the strength of the girder to panel connection to differ from a typical composite girder interface because of the grouted haunch, multiple interface planes and the fact that the shear connectors were grouped in isolated pockets. His experimental approach looked to address these concerns. He conducted 36 push-off tests which represented a precast deck being connected to a precast concrete girder. The deck and girder specimens were identically shaped with the shear studs attached to one directly and then grouted to the other. See Figure 22

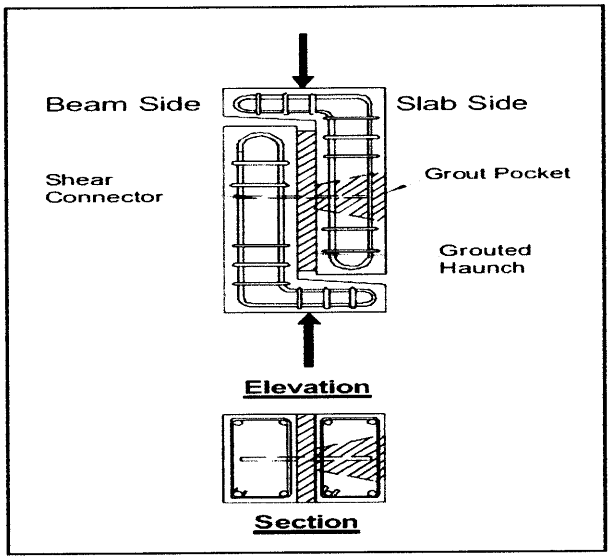


Figure 22: Menkulasi Test Setup (Menkulasi 2005)

Menkulasi designed his testing program to investigate the following four parameters: 1. The type of shear connector would include no connector, extended stirrups, post-installed reinforcing bars and insert anchors. 2. The cross-sectional area of shear connector. 3. The type of grout used. 4. The grouted haunch height used included 1, 2, and 3 inch high haunches. (Menkulasi, 2005)

For the testing setp and procedure, Menkulasi placed the assembled specimen in a specifically designed testing frame. This frame allowed the girder side of the connection to slide with respect to the slab side, which was itself fixed in place. The beam side was supported on four steel pipes. These pipes allowed the beam and slab to slide relative to each other with the application of a shearing force. Additionally, a normal force was applied to the specimen. This normal force was used to simulate the self weight of the deck. The normal force applied was 2.5 kips. Once the normal force was applied to the slab side of the specimen, the shearing force was then applied. Loading was increased until a crack formed. The normal force was adjusted as the specimen pushed upwards and cracking started. The normal force was returned to 2.5 kips, and shear loading continued until 1 inch of slip had been measured between the slab and beam sections. (Menkulasi, 2005)

Menkulasi looked at several different results from these 36 iterations. His result analysis included: load-slip behavior, load-strain behavior, comparison of grout types, comparison of haunch heights, and alternate shear connectors used. He also did extensive strength predictions and compared those strengths to accepted design equations. A brief look at each of his result analysis will be presented here.

Menkulasi found that with an increasing shear connector diameter, the peak load increased. After the peak load, cracking always appeared along one of the interfaces. While the maximum load was not reached again, a lesser load could be resisted through large relative slippage. The cracking always appeared between the grout and the beam or slab. With a one inch deep haunch, the crack always appeared on the beam side, but the location of the crack varied in location with a varying haunch height. (Menkulasi, 2005)

Menkulasi looked at the load-strain behavior of each of his specimens. The strain in the studs approached, but did not cross the yield strain of the material, before cracking appeared. After cracks in the surrounding grout appeared, he observed that the strain immediately became more than the material yield strain. See Figure 23 for a typical representation of his load-strain results.

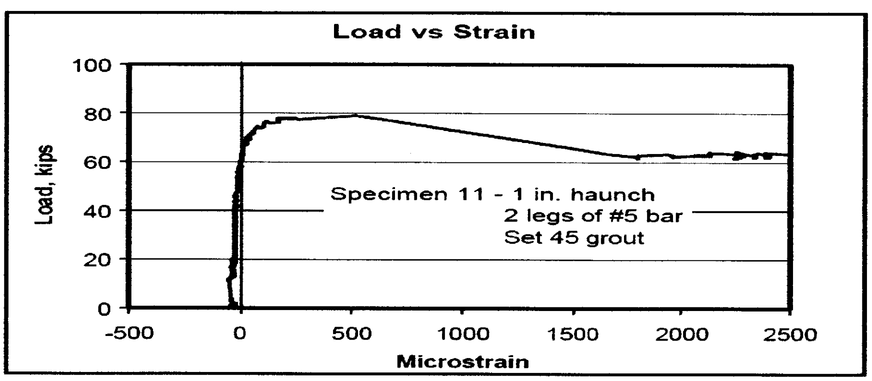


Figure 23: Load vs. Strain Relationship for a Typical Experimental Run (Menkulasi, 2005)

Menkulasi compared two different grout types, latex modified and Set 45, in 12 of his specimens. He compared the peak shear stress vs. clamping stress in three different shear connector arrangements with the two grouts. The shear connector arrangements werethe following: 1. No connectors 2. Two legs of No. 4 rebar 3. Two legs of No. 5 rebar. For his analysis the clamping stress was defined as $v_{clamp} = (A_{vh}f_y + P_n)/(b_v s)$. He defined the peak shear stress as: $v_{peak} = V_{max}/(b_v s)$. (Menkulasi, 2005) See Figure 24 for his results of different grout types.

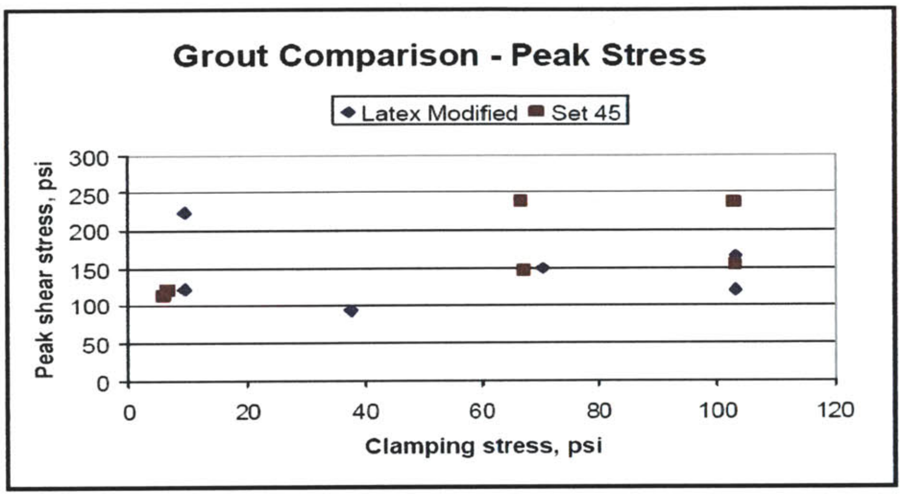


Figure 24: Comparison of Different Grout Types vs. Peak Stress (Menkulasi, 2005)

In his comparison of haunch heights, Menkulasi used two different haunch heights in order to determine how the height of the haunch influenced the shear strength of the connection. He used the same area of steel reinforcement throughout the haunch height tests. He used a haunch height of one inch and three inches. He tested these haunch heights with the following reinforcement configurations: 1. No connectors. 2. Two legs of No. 4 rebar. 3. Two legs of No. 5 rebar. 4. Four legs of No. 4 rebar. He once again compared the clamping stress to the peak shear stress of the different connectors. He found no significant difference in shear strength with differing haunch heights. See Figure 25.

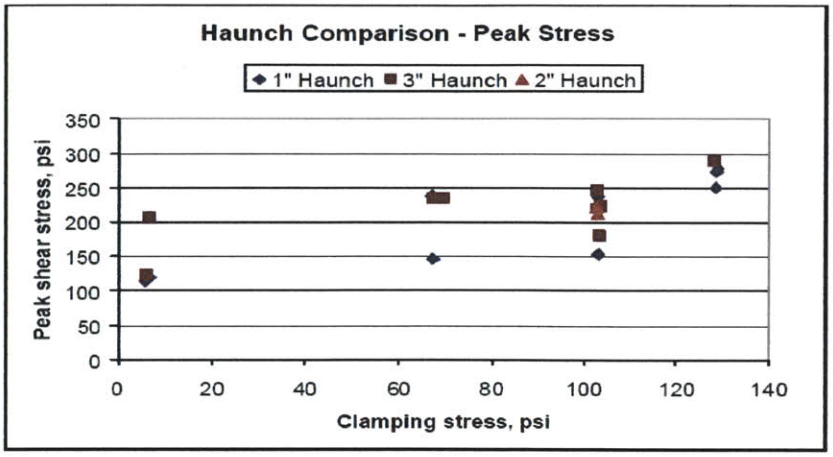


Figure 25: Comparison of Different Haunch Heights vs. Peak Stress (Menkulasi, 2005)

In Menkulasi's final area of investigation, he examined the use of various different shear connectors. A typical bridge will use headed shear studs that are welded to a steel girder or hooked rebar that is embedded in a concrete girder. He used three different types of alternate shear connectors including: 1. Post-installed hooked reinforcing bars (See Figure 26) 2. Dayton-Richmond anchors (See Figure 27) 3. Shear Keyed specimens (See Figure 28)

Menkulasi found that the post-installed reinforcing bars were very simple to install. After he poured the girder, he drilled a hole in the girder and installed the reinforcing bars with epoxy. For the Dayton-Richmond anchors, the installation was a two step process. First, he placed the looped portion prior to the pouring of the girder. After the girder was in place, he installed the ¾ inch coil bolt. He formed the shear keys into the girder during the pouring process. He found from the load vs. slip comparison of the three alternate shear connectors that all were a viable solution for transferring shear. The results of the tests from the hooked reinforcing bars, Dayton-Richmond anchors and the shear keys can be seen in Figures 29, 30, 31.

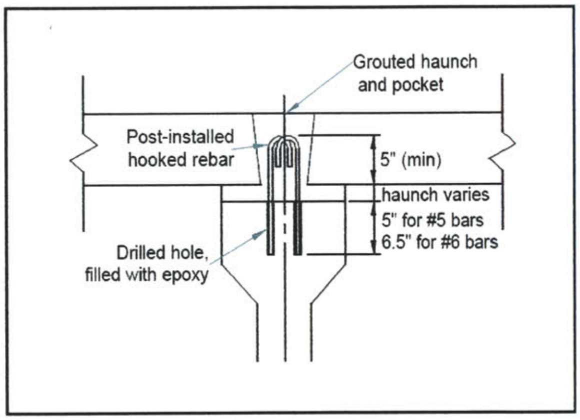


Figure 26: Post-Installed Hooks (Menkulasi 2005)

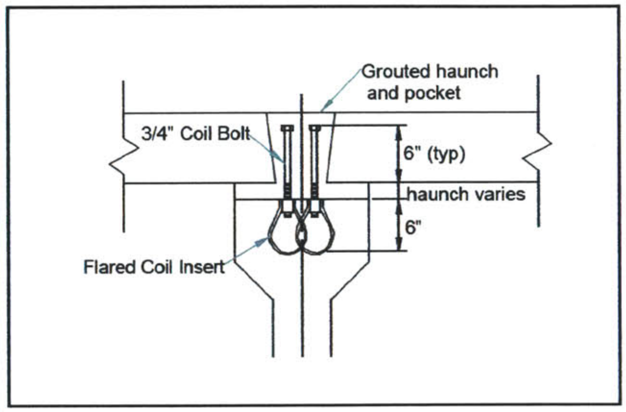


Figure 27: Dayton-Richmond Anchors (Menkulasi 2005)

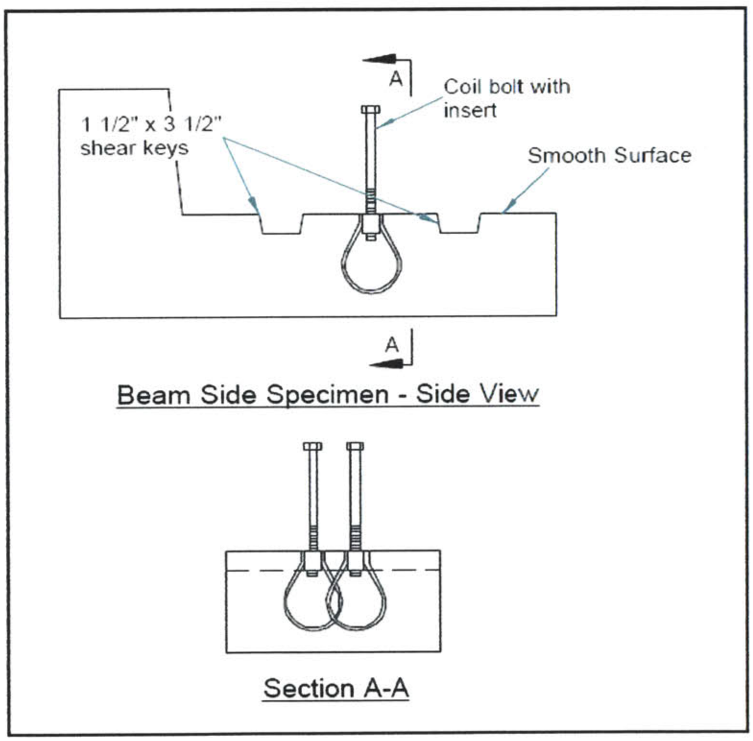


Figure 28: Shear Keyed Specimens (Menkulasi 2005)

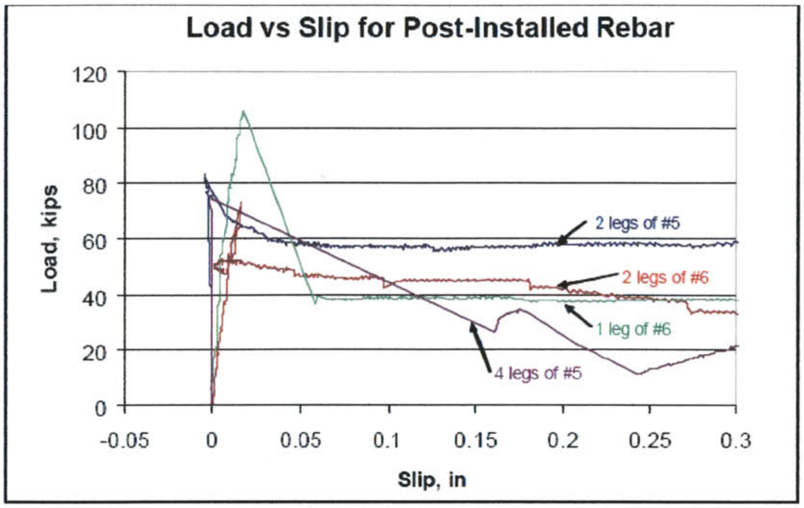


Figure 29: Post-Installed Rebar Load vs. Slip (Menkulasi 2005)

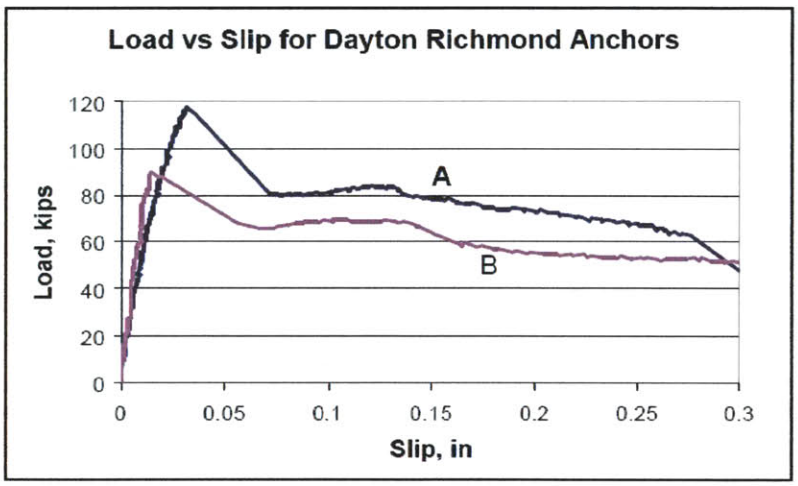


Figure 30: Dayton-Richmond Anchors Load vs. Slip (Menkulasi 2005)

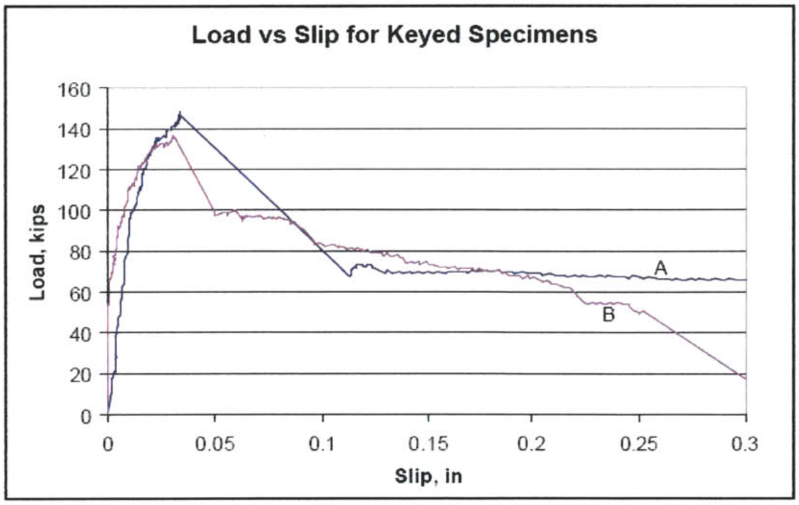


Figure 31: Keyed Specimen Load vs. Slip (Menkulasi 2005)

Menkulasi found, that of the ACI 318-02, AASHTO Standard Specifications for Highway Bridges and the AASHTO LRFD Design Specifications, the AASHTO LRFD Specifications most accurately predicted the loads that he found in his experimentation. He felt that the un-conservative values of the ACI 318-02 and AASHTO Standard Specifications compared to his research could be attributed to the fact that those codes were for new concrete cast against old concrete while his experiment included all precast concrete elements. See Figure 32.

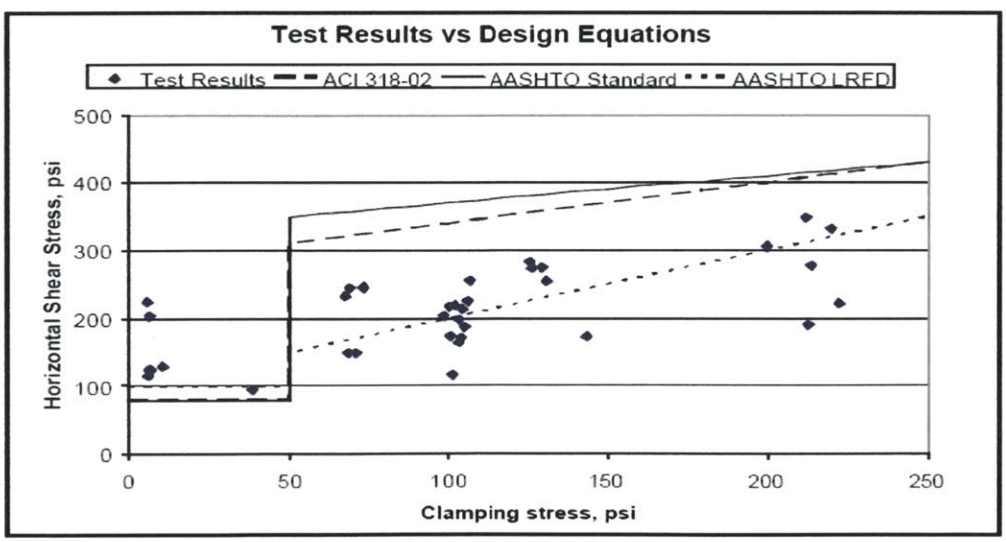


Fig. 20. Peak stresses from push-off tests compared to design equations.

Figure 32: Comparison of Peak Stress from Experiments vs. Design Equations (Menkulasi 2005)

From the experimental program, Menkulasi found six primary conclusions:

1. The Set 45 formulation developed slightly higher peak shear stresses than the latex modified grout.
2. There was no significant difference in peak shear stress between specimens with one and three in. haunch heights.
3. The extended stirrups must be detailed to have a minimum of five inch embedment into the deck panel.
4. The alternate shear connectors investigated in this study are viable for use with the precast panel system.
5. In addition, equations have been presented to quantify the shear stress that cuased cracking at the interface between the two elements and to quantify the stress that can be carried across the cracked interface. Among the current codes and specifications, the equation in the AASHTO LRFD Specifications is the best predictor of the peak interface strength.
6. Designers of this type of clustered connection must prevent a cone creakout type of failure that may occur if many connectors are clustered together in one location or if the embedment is inadequate. Proper embedment and spacing of connectors will ensure yielding of the steel and ductile behavior of the interface. (Menkulasi, 2005)

2.7 Oehlers

The previous bridge deck systems and connection tests all utilized some form of shear connection between the bridge deck and the supporting girders. The shear connection makes a bridge deck and girder system capable of being much lighter, because the deck is used in conjunction with the girder to support the loads. This means that under service loads, the shear connection must not fail or the strength of the system will be greatly reduced. Much research has been done on these shear connections with the focus of ensuring they maintain their strength throughout the service life of a bridge.

Oehlers, et al. conducted extensive research aimed at predicting the static strength of stud shear connectors in composite beams. He found that the functional relationship for the static strength of the shear connection was based on the geometry of the shear stud, specifically its cross sectional area, and on the stiffness and strength of the other materials used in the connection, namely the concrete surrounding the stud. (Oehlers, 1987)

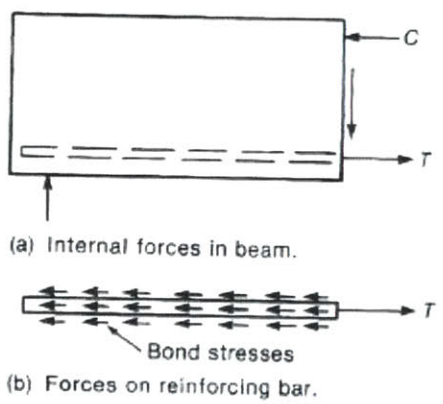
Oehlers found that the difference between the strengths of connectors in composite beams and laboratory studied push-off specimens was the normal force applied to composite beams across the steel beam and concrete interface. Oehlers conducted a set of experiments on push-off specimens with varied compressive forces in order to study the affect of that normal force on the shear strength of the connection. Oehlers found that the removal of the normal force on the specimens results in a 19% reduction in shear strength from the Lehigh tests. He also noted, that due to the plasticity of steel studs, beams with few studs should have the design strength of the studs reduced.

Oehlers found that the static failure load of the stud shear connections in composite beams could be determined by $P_p = KA(\frac{E_c}{E_s})^{0.40} f_{cu}{}^{0.35} f_u{}^{0.65}$ with $K = 4.1 - n^{-0.5}$ where n is the number of studs in the connection. (Oehlers, 1987)

Oehlers, et al. also have done extensive research on the results of fatigue loading on the strength of stud shear connections. The fatigue load strength of a shear stud connection is based on the static strength of a fatigue stud connection. Through experimental study, Oehlers, et al. concluded that, starting at a peak value of the static strength of the stud shear connector, the fatigue strength decreases as soon as a fatigue load is applied to the connector. The fatigue methodology that he created from these experiments could be used in both a design and analysis method. For design he used, $Q_{of}{}^{5.1} = [10^{\left(3.12-\left(\frac{0.70}{\sqrt{n}}\right)\right)}(1 - \frac{Q_o}{Q_{of}})]/F$ and for analysis he used $Q_{st}{}^{5.1} = [10^{\left(3.12-\left(\frac{0.70}{\sqrt{n}}\right)\right)}(1 - \frac{Q_o}{Q_{st}})]/F$. Both of these equations use an iterative approach to find the nominal strength for the shear connectors. (Oehlers, 1995) Oehlers' work on both static loads of studs and fatigue loading is the basis for the AASHTO design equations used by Badie, Tadros and Menkulasi in their research studies.

2.8 Bond Development in Concrete

The development of bond between reinforcing steel and concrete has been studied extensively. Since reinforcing steel is used in the tension side of beams, the bond between the steel and concrete can be expressed in terms of the tensile forces used in beam design. To test how this force affects the bond, pullout tests have been conducted by numerous researchers. The pullout tests have been used to study the following areas(but not limited to): damage to the surrounding concrete, how initial cracks in the concrete affect development length, length of bar embedment required for full development, and the affect of corroded or deformed steel.



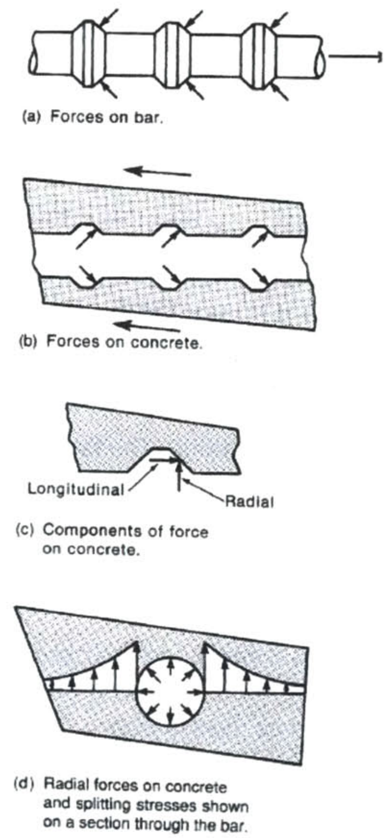


Figure 33: Forces on Reinforcing Bars (MacGregor, 1997)

Figure 34: Forces on Reinfocment and Concrete (MacGregor, 1997)

For example, Xia, et al. studied the affect of corrosion on stirrups used to reinforce concrete beams on the shear performance of the beams. The researchers were able to correlate the amount of corrosion on the shear stirrups with the corrosion induced crack width in the beams and subsequently determine the residual shear strength of the beams. (Xia, 2011)

2.9 Analysis of Literature Review

While reviewing the literature there appears to be no reference to the bond of a steel shear stud to the surrounding grout or concrete when a shearing force is applied.

The literature review has shown that numerous research efforts have attempted to make a bridge more repairable, more replaceable or better able to use alternative connections that increase construction safety. The major projects conducted in the literature review mostly focused on creating a new system or greatly expanding a current system. However, none of the researchers studied, in great detail, the inherent connection used to create composite action between a deck slab and a steel girder, namely the connection between the steel shear stud and the grout or concrete around it. The permutations of the reviewed experiments all used fully bonded and headed studs. The work of Yamane and Tadros are the only times that headless studs are mentioned. While neither study focused solely on the headless studs, they both through experimentation (Tadros) or finite element modeling (Yamane) showed that the headless studs could be used in some capacity. The headless studs they studied were always used in conjunction with headed studs.

While Yamane used headless studs in his model, his analysis did not fully cover how those studs interacted with the grout that enclosed the studs. He assumed, correctly, that the headless nature of the studs only allowed them to transmit horizontal forces. He overcame this by placing headed (bolted) studs in concert with headless studs to allow one to give the desired uplift resistance while the other only transferred the horizontal shear forces.

Researchers built full scale or simulated full scale (through quarter and half scale) bridges and made observations on the whole system scale. In order to make a bridge like a model car similar to the proposal of Tadros et al., it is necessary to fully understand the individual connections that will allow the bridge to "snap" together.

These full scale and quarter scale bridge tests, along with the AASHTO design equations, use the research of Oehlers et al. in order to account for the strength of the shear stud connection. The work of Oehlers clearly shows how the shear connection between a stud and the material surrounding it creates composite action between a bridge beam, or girder, and the bridge deck above it. This research was based on the use of headed shear studs and the fully bonded nature between the headed stud and the material (grout or concrete) surrounding the stud.

In cases of rapid construction, the use of precast prestressed panels is now the cutting edge of technology. This means that the reinforcement that surrounds the shear pockets in the precast panels can be closely designed and constructed in a controlled environment. In order to remove the panels in the future, it is necessary to use a shear connection that can be readily removed. However, more studies are needed to better understand the removability of an individual shear connection, because very limited research has been conducted at this point in time. Although, numerous studies using a shear connection to create composite action, at the individual scale (Oehlers et al.) and the system scale (Tadros, Issa, Badie, Yamane), have been conducted, no one has yet studied how to replace an individual shear stud connection.

3. Analysis of Headless Stud Shear Connection

3.1 Motivation

Using a car to model how a bridge should be built is not a new idea. Tadros et al. mentioned that they desired to make their bridge system like a model car with snap together parts. However, the inherent problem of working with concrete is that the material properties are not constant and need to be accounted for at all times. In order to build a bridge like the cars that drive on it, it is only necessary to consider certain aspects of how a car is built. The key aspect of a car that differs from a bridge is that a car consists of a large number of connections and each of those connections can be taken apart. So in order to make a bridge more like a car, we must look at the individual connections that join the parts of a bridge.

Obviously, studying each individual connection of a product that is inherently made up of materials with changing properties is an overwhelming task. However, looking at the key connections that make bridges lighter and more durable, yet at the same time less repairable is a good place to start. The main connection that makes bridges lighter and less repairable, is the use of shear studs to create composite action between a bridge deck and the girder that supports it. The bridge deck to girder connection is often one where a steel girder supports a concrete deck. Due to recent advances in bridge deck design, the shear stud is connected to the deck through the use of grout. To better understand the individual connection of a grouted shear stud to a steel beam, an experiment on an isolated, grouted stud was conducted. Details are discussed below.

3.2 Experiment

To further analyze the connection between the shear stud and the grout, the size of the stud and strength of the grout were scaled down. The observations from these smaller connections can then be applied to full scale shear connections. Both the grout strength and stud size were altered to account for the interaction between the two materials in a shear connection as found by Oehlers et al. (Oehlers,

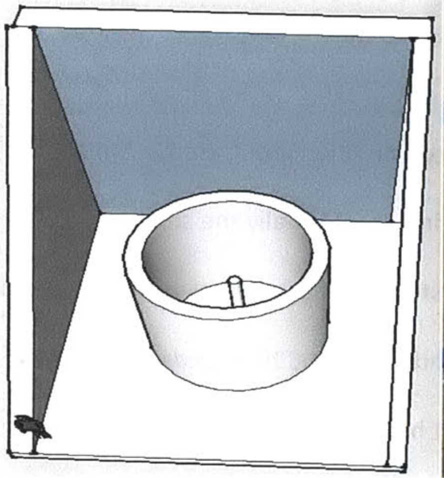
1987) In addition, all aspects of the setup apparatus were over designed to allow for future iterations with larger studs and stronger grout.

3.2.1 Setup

To isolate the shear stud connection, a loading frame was developed that allowed the shear connection to be loaded without the use of a quarter or half scale bridge segment. The frame consisted of three half-inch thick steel plates that were formed into a half box loading frame with diagonal sides. (See Picture 6) This setup allowed observation of the shear load while not impeding the movement of the shear connection.

The stud was welded to an additional half-inch thick steel plate, centered within a 2.5" inner diameter cylinder with half inch thick walls, and then grouted. All eight specimens were poured on the same day and allowed to cure for 28 days.

Welding the stud to a separate steel plate from the loading frame allowed for multiple iterations to be prepared and tested rapidly. The separate test specimens were attached to the loading frame so that the supporting steel plate bore on the back of the loading frame during load application. This arrangement ensured that the shear was fully transmitted through the cylinder to the stud. Since the steel plates, cylinder and loading frame were over designed, they simply supported and encased the grout and steel shear stud while not affecting the loading seen by the grout and stud.



Picture 5: Schematic of Experiment Showing Shear Stud   Picture 6: Experimental Setup

3.2.2 Materials

3.2.2.1 Shear Studs

The studs used in this experiment consisted of ¼ inch diameter smooth, headless steel studs. Headless studs were specifically chosen for several reasons. The use of headed studs is well known and extensively studied. However, the use of headless studs, while previously used in bridge deck panel system designs, has not been extensively studied. The use of headless studs would allow a bridge deck to be more removable, as suggested by Yamane. To fully understand the shear resistance of a headless stud, the headless stud must be studied to the same degree as headed studs.

3.2.2.2 Cylinders

The cylinders consisted of three-inch diameter, six-inch tall cylinders with ½ inch thick walls. The inner diameter of 2.5 inches ensured the shear stud did not contact the wall of the cylinder during load application. The purpose of these cylinders was to allow a shear force application to the stud and grout combination without causing damage at the interface between the grout and the cylinder. The cylinder acts as a containment vessel as well as protecting the stud and grout connection. This protection allows the experiment to focus solely on the interaction between the grout and stud interface.

3.2.2.3 Grout

The grout used was much less rigid than the high strength construction grout used in typical highway bridge applications. The smaller overall scale of the experiment, especially the smaller diameter shear stud, meant that a stronger grout would cause the stud to shear off, making it difficult to study the stud and grout interaction. The grout consisted of 5 pounds of water, 10 pounds of cement and 45 pounds of sand. Additional water was added until the grout became workable. Cylinders were cast when the shear test iterations were poured. The cylinders were broken on the 2nd day of testing for the shear tests. The average results for the three cylinders were f_c'=1.819 ksi and E_c=797.65 ksi.

3.2.2.4 Debonding

In order to test the affect of the bond on the steel stud and the surrounding grout, four of the specimens were tested under a debonded condition. A rubber sleeve was applied to the steel stud and heated to shrinkwrap the rubber to the stud. This application of rubber effectively caused the grout to be unable to bond to the stud. The thin layer of rubber did not add significant strength to the grout causing the system to act as a fully debonded connection. The application of the rubber sleeve took approximately 10 seconds per stud due to the simple application process.

3.2.3 Iterations

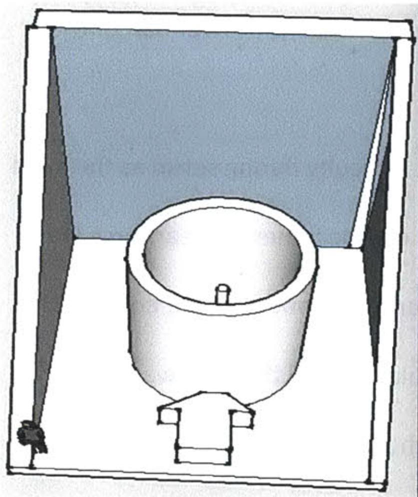
The experiment had two main objectives. First, the experiment examined how the level of bond between the steel stud and the grout that encased it affected the shear strength of the stud connection. Second, the experiment observed the effectiveness of purely headless studs. The experiment consisted of eight total iterations with two iterations of each variation. The grout was the same for all of the iterations. The four variations of the experiment included: 1. Fully Bonded Stud (FB) 2. Fully Debonded Stud (DB) 3. Partially Debonded Stud (DB.5) 4. Partially Sand Encased Stud (S.5).

For the third variation, Partially Debonded Stud, the rubber sleeve was only applied to the lower half of the stud. This caused the upper half of the stud to be fully bonded while the lower half of the stud was fully debonded.

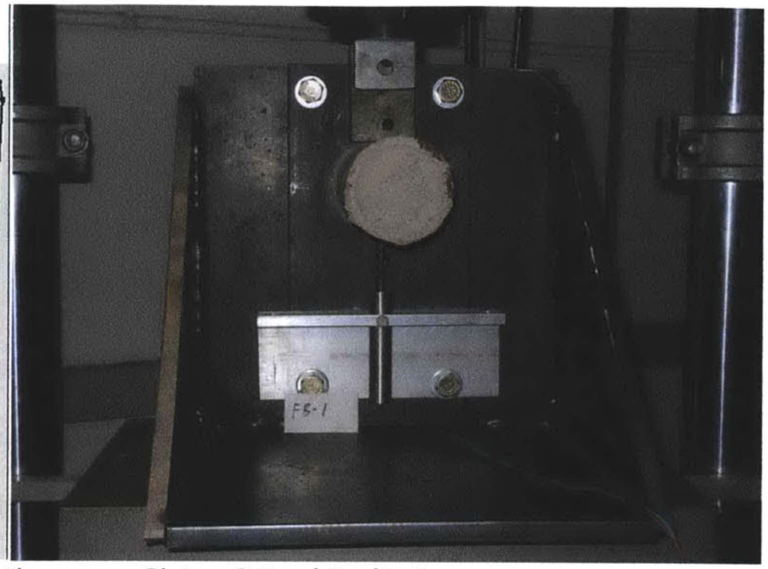
The fourth variation, Partially Sand Encased, caused the most difficulty during setup as the sand had to be contained so that it would not simply empty from the steel cylinder when a shear force was applied. In order to prevent the sand from emptying, the stud was partially grouted at the bottom to approximately one-third of the way up the stud. The sand was then placed until the stud was fully covered and then wetted. The rest of the cylinder was then filled with grout to keep the sand fully contained.

3.2.4 Testing Procedure

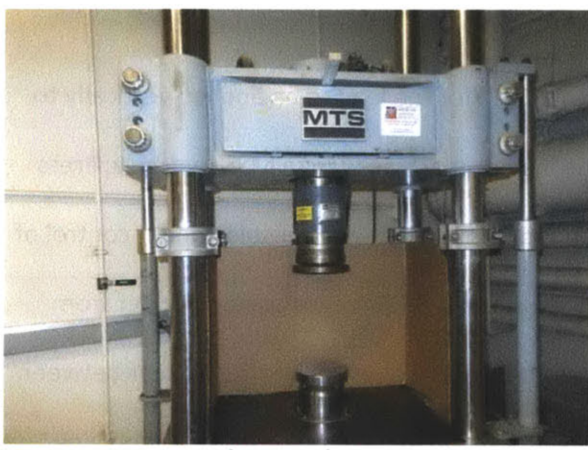
The cylinder was set horizontally in the loading machine for the load to be applied vertically to the base of the steel cylinder. (See Pictures 7 and 8) The loading machine was an MTS Hydraulic Press Machine and the load application was quasi-static. The load was applied with a displacement control of 0.05 inches per minute. The load was applied vertically to prevent the influence of the cylinder from increasing the interface shear. The self weight of the cylinder would cause the interface shear between the cylinder and the bearing plate to influence the recorded strength of the connection. By applying vertical loading, the interaction between the stud and the grout is further isolated. An LVDT was placed under the steel cylinder to measure the movement of the cylinder with load application. The voltage range of the LVDT was set to plus or minus 2.5 volts.



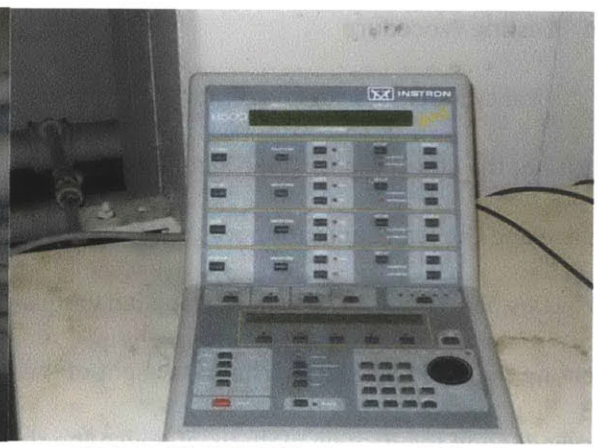
Picture 7: Schematic of Load Application



Picture 8: Load Application



Picture 9: MTS Loading Machine



Picture 10: LVDT Displacement Monitor

4. Results

4.1 General

4.1.1 Fully Bonded

Fully Bonded-1 (FB_1). The specimen was poured on 24 November 2011, and the shear test was conducted on 4 January 2012. The load was applied at a rate of 0.05 inches per minute. The cylinder was observed to lift-off $\frac{1}{8}$ inch within the first moments of load application. The lift-off seemed to occur during the 0.02 inches of movement expected before the shear stud engaged. The lift-off was only observed on the side of the specimen where the load was applied. Once it reached 0.125 inches, the lift-off did not change in magnitude for the duration of the test. The specimen was loaded until the shear stud failed in shear at a load of 1.384 kips. Damage to the grout was observed to have a maximum width of 1.5 inches and pullout of the shear stud was measured to be $\frac{5}{8}$ inch. The damage to the steel shear stud was a complete shear failure.

As this test was the first conducted, the supporting frame was observed to lift off from the loading machine at the point furthest from the load application. To ensure this did not happen in the future, the front end of the supporting frame was clamped to the loading machine for all future iterations. This uplift did not seem to affect the load resisted, damage or movement in any significant manner when the results of this test were compared to the next Fully Bonded test.

Fully Bonded-2 (FB_2). The specimen was poured on 24 November 2011, and the shear test was conducted on 6 January 2012. The load was applied at a rate of 0.05 inches per minute. The cylinder was observed to lift-off $\frac{1}{8}$ inch within the first moments of the load application. The lift-off was similar to FB_1 in that it occurred at the start of the test, was only observed on one side of the cylinder where the load was being applied, and once at $\frac{1}{8}$ inch, the lift-off did not change for the duration of the test. This specimen was loaded to failure of the shear stud in shear at a load of 1.604 kips. Damage of the grout was observed to have a maximum width of 1.5 inches, and pullout of the shear stud was measured to be $\frac{11}{16}$ inch. The damage to the steel shear stud was a complete shear failure.

4.1.2 Fully Debonded

Fully Debonded-1 (DB_1). The specimen was poured on 24 November 2011, and the shear test was conducted on 4 January 2012. The load was applied at a rate of 0.05 inches per minute. The cylinder was measured to have lift-off of $\frac{3}{8}$ inch. The lift-off happened in the first moments of the test, presumably during the period before the shear stud fully engaged. Once the lift-off occurred, it remained at the same amount throughout the duration of the test and was observed to be nearly perpendicular to the supporting steel plate. Unlike FB_1 and FB_2, the rest of the specimens all lifted off directly away from the support, not just on the side of the load application. DB_1 was loaded until the loading reached the physical movement limit due to the nature of the loading setup. All future iterations were loaded to the same physical limit. The specimen did not reach shear failure by the end of the loading. Damage to the grout was observed to have a maximum width of $1\frac{11}{16}$ inches, and pullout was measured at 1 inch. Damage to the steel shear stud was observed as cracking on the same side of the shear stud as the load application. The observed crack went one-quarter of the way through the shear stud.

Fully Debonded-2 (DB_2). The specimen was poured on 24 November 2011, and the shear test was conducted on 6 January 2012. The load was applied at a rate of 0.05 inches per minute. The cylinder was measured to have lift-off of $\frac{3}{8}$ inch. The lift-off was similar to DB_1 in that it was equal around the entire cylinder and not just in the direction of loading like FB_1 and FB_2. The lift-off remained constant throughout the duration of the test. DB_2 was loaded until the machine reached the physical movement limit due to the nature of the loading setup. The specimen did not reach shear failure by the end of loading. Damage was observed to have a maximum width of $1\frac{9}{16}$ inches, and pullout was measured at 1.25 inches. Damage to the steel shear stud was observed on the same side of the shear stud as the load application. The observed crack went one-third of the way through the shear stud.

4.1.3 Partially Debonded

Debonded.5-1 (DB.5_1). The specimen was poured on 24 November 2011, and the shear test was conducted on 4 January 2012. The load was applied at a rate of 0.05 inches per minute. The cylinder was measured to have lift-off of $\frac{1}{4}$ inch. The lift-off was observed to behave in the same manner as DB_1 and DB_2; it remained constant throughout the duration of the test. DB.5_1 was loaded similar to DB_1 and DB_2. The specimen did not reach shear failure by the end of loading. Damage to the grout was observed to have a maximum width of 2.5 inches, and pullout was measured at 1 inch. No damage was observed to the steel shear stud.

Debonded.5-2 (DB.5_2). The specimen was poured on 24 November 2011, and the shear test was conducted on 6 January 2012. The load was applied at a rate of 0.05 inches per minute. The cylinder was measured to have lift-off of $\frac{3}{8}$ inch. The lift-off was observed to behave in the same manner as DB_1, DB_2 and DB.5_1. The lift-off remained constant throughout the duration of the test. DB.5_2 was loaded similar to DB_1, DB_2 and DB.5_1. The specimen did not reach shear failure by the end of loading. Damage to the grout was observed to have a maximum width of 1.5 inches, and pullout was measured at 1.25 inches. No damage was observed to the steel shear stud.

4.1.4 Partially Sand

The two Partially Sand iterations showed similar, yet very different behavior when subjected to a shearing load. The Partially Sand iterations exhibited the same behavior in regards to the initial peak load of the specimen. Once the shear stud engaged, the peak load was observed before the bond between the grout and stud was broken. For the Partially Sand specimens, the only bond available to be broken was that of the grout at the base of the stud that was placed to ensure the sand did not leak during the experiment. Once the specimen had broken the bond, the initial decline in strength was similar to the other specimens. Unlike the other partially bonded specimens, the Partially Sand specimens reacted similar to the fully bonded specimens after an additional 0.2 inches of relative slip. The specimens' strength reduction leveled off in a similar manner until about 0.5 inches of relative slip. At this point, the Partially Sand specimens began to regain strength in the connection. (See Graph 4) No damage was observed to the steel shear stud.

Sand.5-1 (S.5_1). The specimen was poured on 24 November 2011, and the shear test was conducted on 4 January 2012. The load was applied at a rate of 0.05 inches per minute. The cylinder was measured to have lift-off of $\frac{1}{4}$ inch. The lift-off observed was the same as all other debonded specimens; it remained constant throughout the remainder of the test. The specimen was loaded similar to the other debonded specimens, and it did not fail in shear before the end of loading. Damage to the grout was observed to have a maximum width of 2 inches, and pullout was measured at $1\frac{1}{8}$ inches. No damage was observed to the steel shear stud.

Sand. 5-2 (S.5_2). The specimen was poured on 24 November 2011, and the shear test was conducted on 6 January 2012. The load was applied at a rate of 0.05 inches per minute. The cylinder was measured to have lift-off of $\frac{3}{16}$ inch. The lift-off observed was the same as all other debonded specimens; it remained constant throughout the remainder of the test. The specimen was loaded similar to the other debonded specimens, and it did not fail in shear before the end of loading. Damage to the grout was observed to have a maximum width of 2 inches. Pullout was measured at $1\frac{3}{16}$ inches. No damage was observed to the steel shear stud.

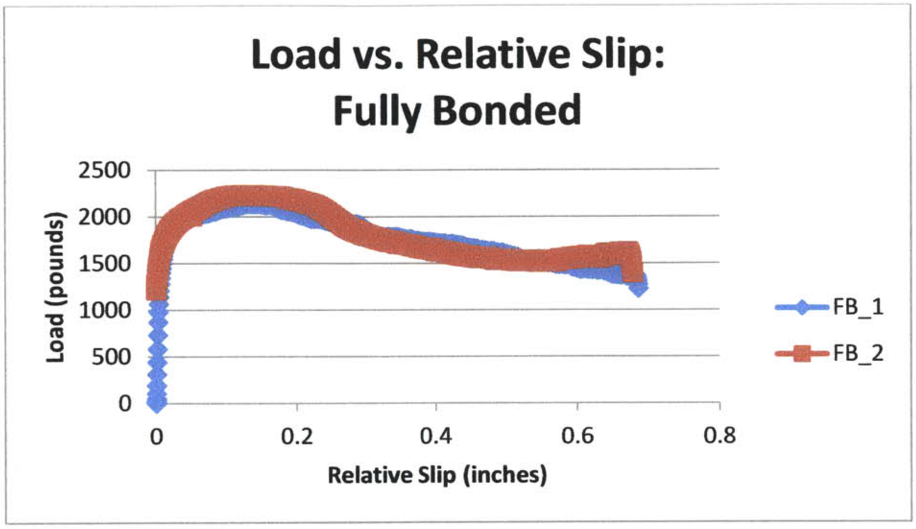
Both of the sand specimens regained lost strength as the relative slip of the specimen increased. Unlike S.5_1, S.5_2 regained strength beyond what was the initial peak strength before the bond was fully broken. This was unexpected and further investigation showed that the load application caused the loading block to push into the supporting steel plate. The extra resistance was not due to an extremely strong shear connection, but to the lack of lift-off, causing the loading block to shear the supporting steel plate. Despite this error in load application, the resulting load versus relative slip curve shows the same shape as the S.5_1 test.

4.2 Load vs. Slip

When a shearing load was applied to the specimen, the resulting load vs. relative slip relationship was both expected and interesting. The headless stud tests resulted in the peak load being achieved at the expected engagement of the shear stud at about 0.02 inches. Since the bond between the grout and the shear stud was expected to break when the stud was engaged, the load was expected to taper off. As expected, the resisted load tapered off rapidly after the peak load was achieved.

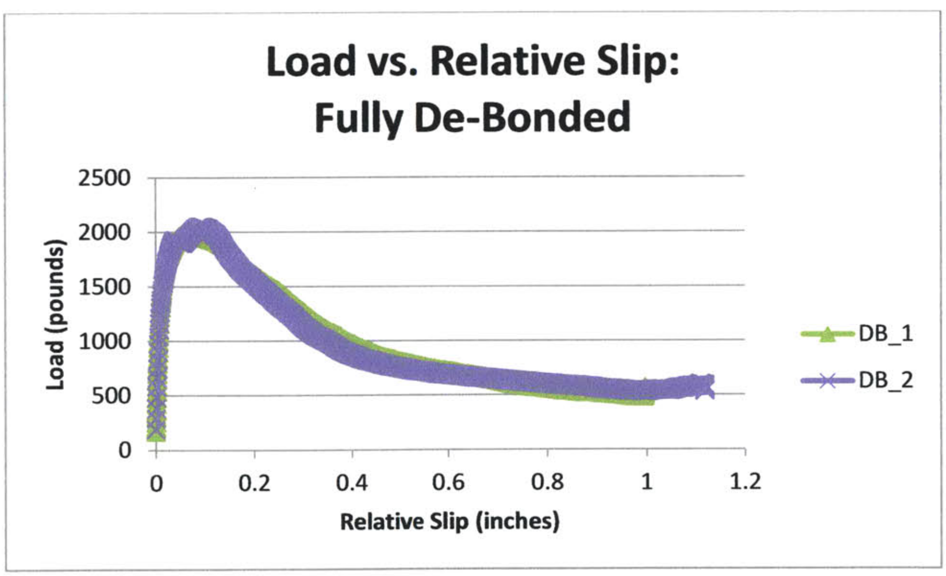
The fully bonded, partially debonded and fully debonded iterations all displayed a continually lessening load resistance after peak load was achieved for the next 0.2 inches of relative slip. With further relative slip, the strength of the connection leveled off or regained some of the lost strength.

See Charts 1-4 for the load vs. relative slip for each iteration. The first fully bonded iteration (FB_1) had a peak load resistance of 2.144 kips and a minimum load resistance of 1.351 kips. The second fully bonded iteration (FB_2) had a peak load resistance of 2.236 kips and a minimum load resistance of 1.515 kips. (See Graph 1)



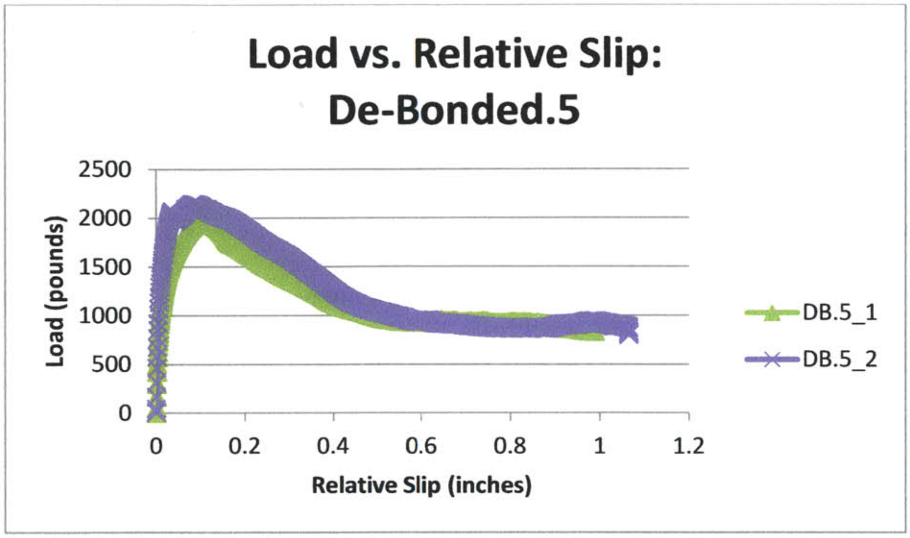
Graph 1: Fully Bonded Iterations Load vs. Slip Comparison

The first fully debonded iteration (DB_1) had a peak load resistance of 1.984 kips and a minimum load resistance of 0.486 kips. The second fully debonded iteration (DB_2) had a peak load resistance of 2.053 kips and a minimum load resistance of 0.528 kips. (See Graph 2)



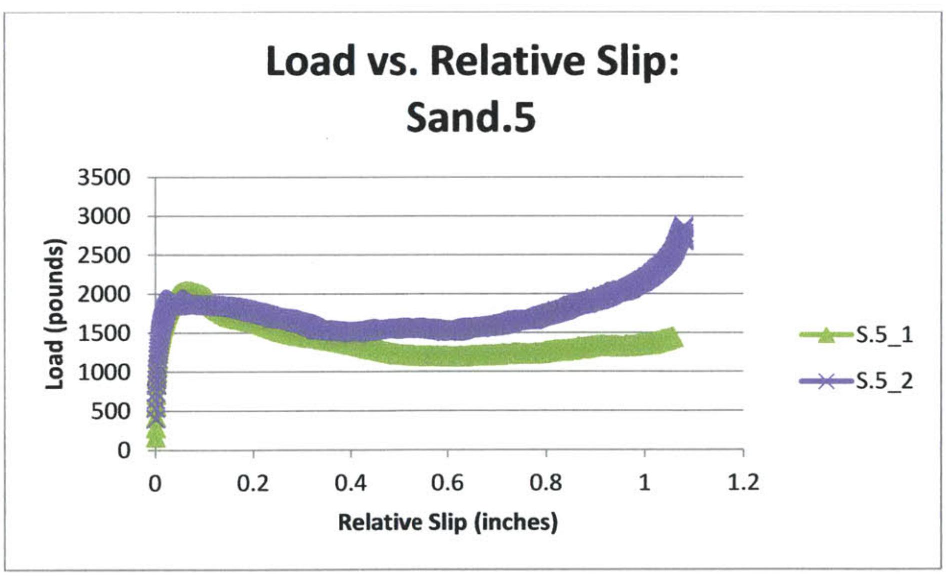
Graph 2: Fully De-Bonded Iterations Load vs. Slip Comparison

The first partially debonded iteration (DB.5_1) had a peak load resistance of 2.003 kips and a minimum load resistance of 0.913 kips. The second partially debonded iteration (DB.5_2) had a peak load resistance of 2.147 kips and a minimum load resistance of 0.865 kips. (See Graph 3)



Graph 3: Partially De-Bonded Iterations Load vs. Slip Comparison

The first sand iteration (S.5_1) had a peak load resistance of 2.040 kips, a minimum load resistance of 1.188 kips, and a residual load resistance of 1.438 kips. The second sand iteration (S.5_2) had a peak load resistance of 1.942 kips, a minimum load resistance of 1.504 kips, and a residual load resistance of 1.673 kips. (See Graph 4)



Graph 4: Sand Iterations Load vs. Slip Comparison

4.3 Damage

Upon inspection after the application of load, the specimens showed a varying degree of damage to the grout at the base of the shear studs and to the shear studs themselves.

4.3.1 Steel Stud Damage

Damage to the steel stud was observed in four of the specimens. The most conclusive damage was to the two fully bonded specimens that were able to be loaded until failure. They were the only specimens that were able to be loaded until failure; the other six specimens all did not fail. This result was not surprising for the fully debonded and partially debonded specimens. The lack of bond, combined with the continually lowering resistance to load in the specimens, indicate that the ductility of the steel would prevent it from failing. However, both partially sand specimens did not fail either. The increased strength of those two specimens indicate that the steel stud should adsorb more load and possibly lead to damage of the steel stud. However, this was not the case. The specimens were able to absorb more load, but did not cause damage in the steel shear stud. See Appendix A for pictures of steel stud damage.

The two fully debonded specimens showed damage to the steel shear stud. It would be expected that, since these specimens had the least bond between the stud and the grout, the stud would absorb the least amount of the load. The fact that these specimens were damaged more than the specimens with more bond (partially debonded and partially sand) was an unexpected result.

4.3.2 Grout Damage

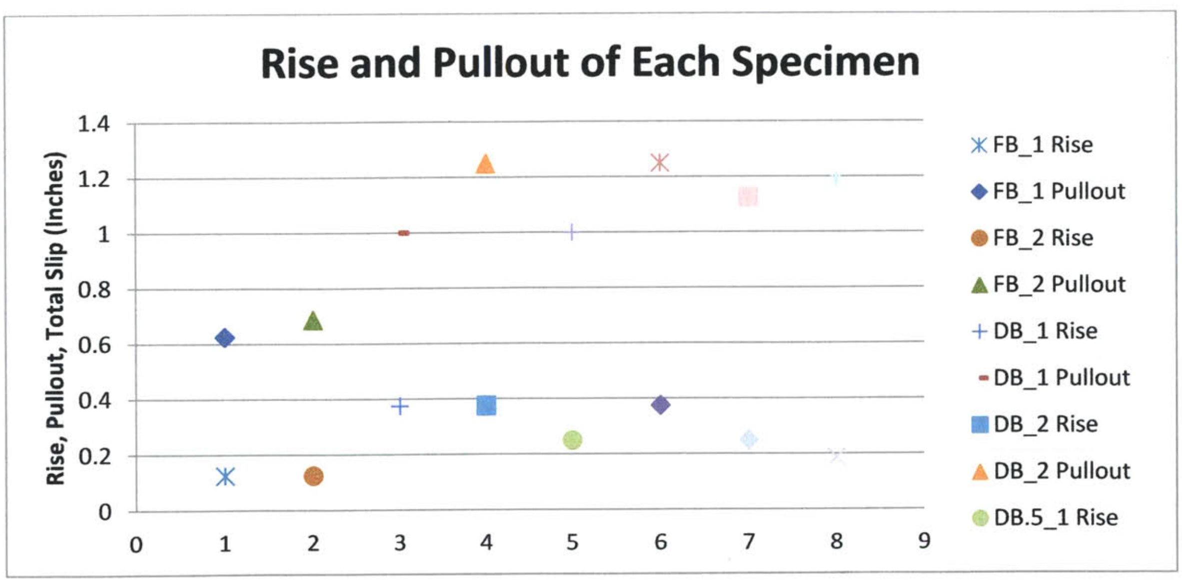
Damage to the grout that occurred on all specimens was inconclusive. The grout at the base of each specimen was damaged, but there was no noticeable pattern throughout the specimens (See Appendix A). Prior to the experiment, I suspected that the half sand specimens would leak sand during application of the load, but this leaking did not occur. Instead, the partially sand specimens all showed a similar type of damage as the other six specimens.

I found no conclusive difference between the damage observed on the eight specimens. The damage appeared to be unrelated to the type of bond applied between the grout and the shear stud. This damage is expected since the stresses at the base of the stud will always be highest in a shearing experiment, regardless of bond. Based on the resulting damage, it seems that the strength of the specimens and the load-slip relationship is solely based on the bond between the shear stud and the grout or other surrounding material.

4.4 Movement of Specimens Relative to original configuration

4.4.1 Pullout of Each Specimen

During the experiment, each specimen separated from the supporting metal plate during load application. While the separation was not instantaneous, it did occur during the first minutes of load application. Once each specimen separated from the supporting steel to a certain extent, which varied for each iteration, the separation remained constant throughout the duration of the experiment. The only specimens not measured were the fully bonded iterations. (See Graph 5) Due to safety concerns and for continuity between test iterations, an accurate measurement could not be taken before the specimen failed with the fracture of the steel shear stud.



Graph 5: Rise (lift-off), Pullout and Total Slip for all Specimens

4.4.2 Rise as a Percentage of Pullout and Total Slip

When the total slip and pullout are compared to the separation from the supporting steel for each specimen, the lowest level of strength was greater for the specimens with rise being a smaller percentage of total slip or pullout. (See Chart 1) When the specimen's minimum load resistance became greater, the separation between the supporting steel (rise or lift-off) is a smaller portion of the pullout of the shear stud. The same comparison is applicable to lift-off versus total slip for each specimen as well. This comparison of movements seems to indicate that the stronger the minimum strength of the connection, the less the uplift forces will act to separate each individual shear connection from the supporting girder. See Appendix A for pictures of the rise of each specimen.

Movement of Cylinder					
Iteration	Rise	Pullout	Total Slip	Rise/Pullout	Rise/Total Slip
FB-1	0.125	0.625	0.689	20%	18%
FB-2	0.125	0.6875	0.68	18%	18%
S.5-1	0.250	1.125	1.06	22%	24%
S.5-2	0.1875	1.1875	1.08	16%	17%
DB.5-1	0.250	1	0.98	25%	26%
DB.5-2	0.375	1.25	1.06	30%	35%
DB-1	0.375	1	0.992	38%	38%
DB-2	0.375	1.25	1.118	30%	34%

Chart 1: Movement of Cylinder Comparison for all Specimens

5. Interpretation of Results

5.1 Peak strengths of Specimens compared to AASHTO 6.10.10.

Issa, et al. suggested that the AASHTO design equation could be over estimating the strength of a shear stud connection by up to 22%. When comparing the peak values of the headless shear studs with any level of debonding to the AASHTO LRFD 2007 equation 6.10.10, the factored peak value of the headless studs is well within the 22% range suggested by Issa, et al. (See Chart 2) It was expected that the peak loads would not be as high as those observed from a headed stud, but it was not expected that the headless studs would achieve a maximum load within the range suggested by Issa. With all the iterations having a maximum load within the 22% range suggests that the AASHTO design equation, along with Issa's observations, could be used to design the number of headless shear studs necessary to resist a design loading. This result also suggests that the level of debonding, while leading to a reduction in the maximum load resisted, does not regulate a debonded stud to a useless state. Studs with some level of debonding or alternate material used to transfer shear can be used to transfer shear loads.

Maximum load (kips) vs. AASHTO 6.10.10 w/ 22% reduction		
Iteration	Max Load	AASHTO -22%
FB_1	1.072	0.729
FB_2	1.118	0.729
S.5_1	1.020	0.729
S.5_2	0.971	0.729
DB_1	0.992	0.729
DB_2	1.026	0.729
DB5_1	1.002	0.729
DB5_2	1.073	0.729

Chart 2: Maximum Load Comparison vs. AASHTO 6.10.10 with 22% Reduction

5.2 Time at peak strengths.

For this analysis, peak strength is defined as the maximum load withstood minus 100 pounds before and after the peak load was achieved. The time each specimen was able to withstand the peak load can be seen in Chart 3 below. The fully bonded specimens, as expected, spent the longest time in the peak load range. The rest of the specimens all ranged from 77 seconds to 195 seconds in the peak range. I expected that the fully debonded specimens would spend the shortest time in the peak load range, but that was not the case. The second partially debonded specimen (DB.5_2) was the shortest time at 77 seconds, followed by the first sand specimen (S.5_1) at 96 seconds. Both fully debonded specimens were over 120 seconds. With the widely varying times spent in the peak load range, it appears that the level of debonding, once the specimen has been debonded at all, does not have a direct correlation on the time spent at the peak load.

Time at peak load (seconds)	
Iteration	Time
FB_1	174
FB_2	207
S.5_1	96
S.5_2	195
DB_1	157
DB_2	122
DB5_1	77
DB5_2	160

Chart 3: Time, in Seconds, that each Specimen Sustained Peak Load

5.3 Minimum strengths of Specimens compared to AASHTO 6.10.10.

Since the experiment found a significant reduction in the peak load resisted to the minimum load (before rupture if rupture happened), it is worthwhile to also compare the 22% reduction of the AASHTO equation to the minimum strength of the shear connections during the course of the loading. While both of the fully debonded and partially debonded specimens were well below the 22% threshold, one of each of the fully bonded and sand specimens were within the 22% range at their minimum load values. (See Chart 4) This load comparison suggests that the use of a headless stud could be incorporated into bridge design, with a headed stud only necessary to restrict the initial uplift of the deck from the girder.

The nominal strength of a shear stud in kips is given in Article 6.10.10 of Section 6 of the AASHTO LRFD as:

$Q_n = 0.5A_{sc}\sqrt{f_c' E_c} \leq A_{sc}F_u$ (Eq. 6.10.10.4.3-1)

with A_{sc}= cross sectional area of shear stud (inches squared), F_u= ultimate strength of shear stud (ksi).

Minimum load (kips) vs. AASHTO 6.10.10 w/ 22% reduction		
Iteration	Min Load	AASHTO -22%
FB_1	0.675	0.729
FB_2	0.758	0.729
S.5_1	0.594	0.729
S.5_2	0.752	0.729
DB_1	0.243	0.729
DB_2	0.264	0.729
DB5_1	0.457	0.729
DB5_2	0.433	0.729

Chart 4: Minimum Load vs. AASHTO 6.10.10 with 22% Reduction

5.4 Ending strength of Sand compared to AASHTO 6.10.10.

The two sand iterations also showed aspects of regaining strength in the shear connection after large relative slips. The sand connections' ability to regain strength to within the 22% range suggests that some form of sanded connection could be used to retain the strength of a bridge despite large relative slips that may accompany damage from a seismic event or other extreme loading. As seen in Charts 2 and 4, while the sand connections do not contribute to the initial shear strength as much as a fully bonded connection, once the fully bonded connection's bond has broken, it fails in a brittle manner. The sand connections, on the other hand, have a rather large amount of ductility. While the relative movement of a girder to the concrete deck is not recommended for normal operations, the ability of the sanded connection to respond with increased shear resistance under abnormal relative movements allows the connection to aid in the shear transfer during extreme loadings. (See Chart 5)

Ending load (kips) vs. AASHTO 6.10.10 w/ 22% reduction		
Iteration	End Load	AASHTO - 22%
S.5_1	0.719	0.729
S.5_2	0.836	0.729

Chart 5: Ending Load for Sand Iterations vs. AAHSTO 6.10.10 with 22% Reduction

5.5 Minimum strength of Specimens compared with AASHTO 5.8.4.

Section 5.8.4 of AASHTO 2007 allows the shear friction theory to be used in the design of a shear stud. Comparing the values of the AASHTO equation shown below, which are based on as-rolled structural steel with concrete anchored with headed studs, to the results of my experiment shows that the use of headless studs does not greatly affect the predicted values for V_{ni}. Proposed values for μ and "c" are given as well. The assumed AASHTO values of μ and "c" are 0.7 and .025 ksi, respectively. As expected, with a reduction in the bond between the steel and surrounding grout, the values of μ and "c" decrease. However, for the fully bonded iterations the experimental values are negligibly less as in FB_1 or greater than as in FB_2 the expected value.

The nominal value in kips of the headed shear stud is the least of the following three equations:

$$V_{ni} = cA_{cv} + \mu A_{vf} f_y \quad \text{(Eq. 5.8.4.1-3)}$$

$$V_{ni} \leq 0.2 f_c' A_{cv} \quad \text{(Eq. 5.8.4.1-4)}$$

$$V_{ni} \leq 0.8 A_{cv} \quad \text{(Eq. 5.8.4.1-5)}$$

with c= 0.025 ksi, μ=0.7, A_{cv}=area of concrete considered to be engaged in interface shear transfer (in^2), A_{vf}=area of shear reinforcement crossing the shear plane (inches squared), and f_y= yield stress of horizontal shear reinforcement (ksi).

Minimum load (kips) vs. AASHTO 5.8.4 w/ proposed values for c and μ				
Iteration	Min Load	AASHTO	proposed μ	proposed c
FB_1	1.351	1.360	0.700	0.025
FB_2	1.515	1.360	0.700	0.025
S.5_1	1.188	1.360	0.610	0.02
S.5_2	1.504	1.360	0.700	0.025
DB_1	0.486	1.360	0.260	0.003
DB_2	0.528	1.360	0.280	0.004
DB5_1	0.913	1.360	0.470	0.013
DB5_2	0.864	1.360	0.450	0.012

Chart 6: Minimum Load vs. AASHTO 5.8.4 with Proposed c and μ for all Specimens

Minimum load (kips) vs. AASHTO 5.8.4 w/ proposed values for c zeroed out, then μ altered				
Iteration	Min Load	AASHTO	proposed μ	proposed c
FB_1	0.834	1.360	0.700	0.025
FB_2	0.174	1.360	0.700	0.025
S.5_1	1.188	1.360	0.680	0
S.5_2	1.504	1.360	0.700	0.025
DB_1	0.486	1.360	0.280	0
DB_2	0.528	1.360	0.300	0
DB5_1	0.913	1.360	0.520	0
DB5_2	0.817	1.360	0.460	0

Chart 7: Minimum Load vs. AASHTO 5.8.4 with Proposed c and μ for all Specimens, Zero Out c First

Since this experimental program consisted of changing the level of cohesion between the steel studs and the surrounding grout, the comparison to the design values from AASHTO Equation 5.8.4 can be approached in two different ways. First, as seen in Chart 6, the values for c and μ were reduced simultaneously until the values from AASHTO 5.8.4 equaled the minimum observed loads from the experimental values. The table shows the proposed values for μ and c from each iteration of the experiment. It should be noted that the FB-1, FB-2 and S.5-2 did not warrant adjusting the values of μ and c from the AASHTO equation.

The alternate means of comparing AASHTO 5.8.4 to the experimental values was to reduce the level of cohesion, c, first. Then, if the strength reduction needed to be increased the value of μ was adjusted. For this approach the resulting values of μ increased 0.01-0.07. This approach makes more sense due to the lack of cohesion between the stud and the surrounding grout. (See Chart 7)

5.6 Ending strength of Sand compared to AASHTO 5.8.4

As noted in the previous paragraph, the value for S.5_2 is greater than the AASHTO 5.8.4 equation estimate, which can be attributed to the error in experimentation. When comparing the expected value to S.5_1, the lower bond with the sand results in a lower value for μ and "c". However, when looking at the ending strength of both sand iterations, both of the iterations have an ending value that is greater than the AASHTO 5.8.4 expected value. (See Chart 8)

Ending load (kips) vs. AASHTO 5.8.4		
Iteration	End Load	AASHTO
S.5_1	1.438	1.360
S.5_2	1.673	1.360

Chart 8: Ending Strength of Sand Iterations vs. AASHTO 5.8.4

5.7 Movement

The rise of the shear connection from the supporting steel plate varied for each type of bond. In the stronger connections, the amount of rise was less than in the weaker connections. The result of the fully bonded connections suggests that the amount of uplift that is needed to be resisted by the heads of headed shear studs is not as large as previously assumed. When Yamane et al. studied the headless and headed stud system, they found that a relatively low number of headed studs were needed in relation to the headless studs. It is possible that even fewer headed studs than Yamane suggested are required.

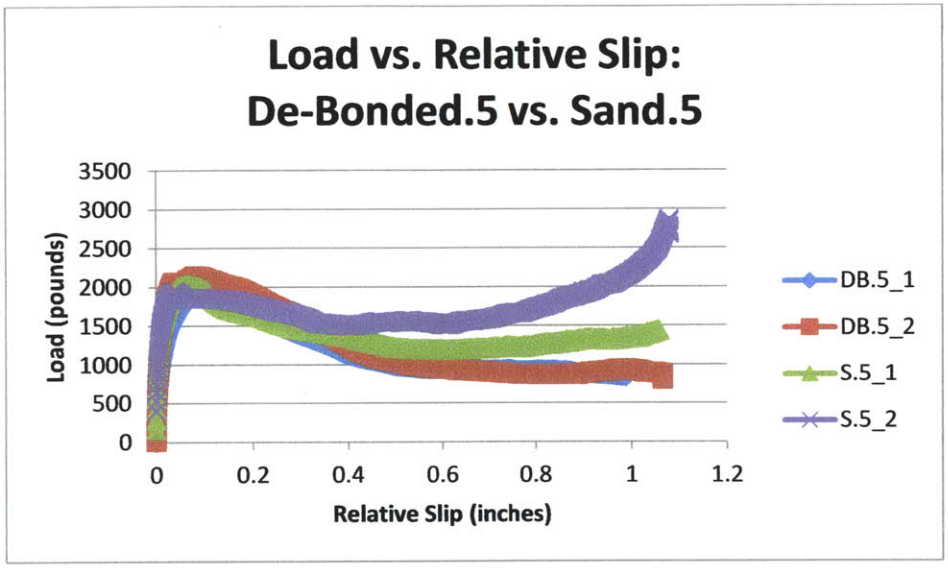
Each specimen exhibited some pullout and rise of the shear connection. The pullout of the shear stud from the grout surrounding it varied for each connection. As expected, the fully bonded specimens had less pullout than the debonded specimens. However, the amount of rise was stagnant after the first moments of load application. This suggests that the pullout was the only movement contributing to the relative slip of the specimen once the rise occurred. Since the debonded connections were not loaded to failure and were loaded over a longer time than the fully bonded connection (the fully bonded connection reached material failure of the shear stud), it is possible that the pullout would have been similar for the fully bonded connection as the debonded connections if the stud had not failed.

5.8 Duration of Loading

The fully bonded connections both failed at approximately the same time in the loading process. The grout around the stud was rigid enough and the bond between the grout and the stud was strong enough to transfer the shear to the stud directly. Conversely, the less bonded connections exhibited much more ductility in their connections. It is not desirable to have the resisted load resisted continually decrease, but each of the debonded studs load resistance did plateau. This minimum value for load resistance suggests that a more ductile connection, that sacrifices approximately 20% of the initial peak strength as well as 30-75% of the sustained strength, can be achieved with some level of debonding on the stud. The application for such a system is especially evident when seismic or other damage type loading is applied to the bridge. The debonded studs would retain some of the strength of a bridge during a seismic or other extreme event. While the fully bonded studns would completely lose their load revisiting capacity through fracturing of the stud.

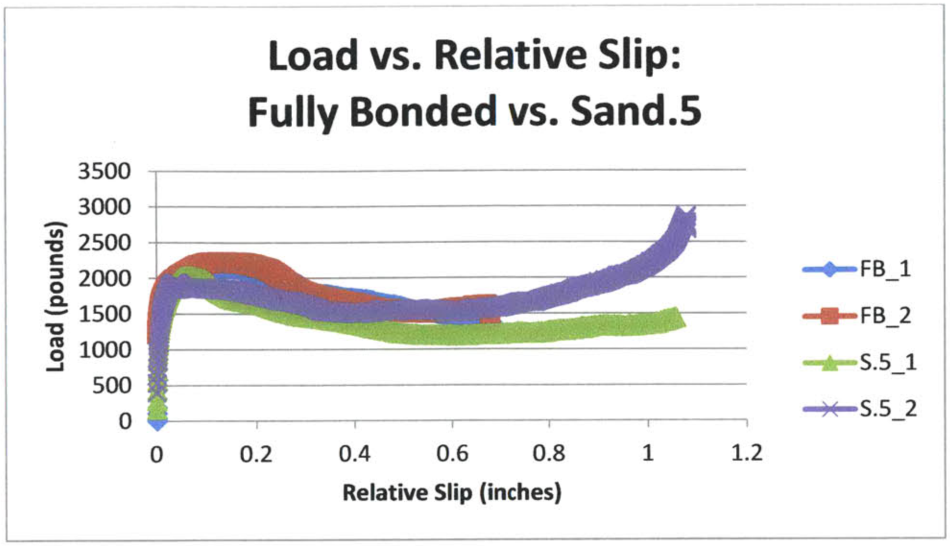
5.9 Sand vs. other materials.

While numerous studies have examined the bond between concrete or grout and reinforcing steel, very few studies have looked at the bond between alternate materials and steel reinforcement. With specific application to a bridge deck and girder system, the use of sand as a component of the shear transfer system has not been looked at previously. The use of a shear connection with a debonded shear stud could achieve a more repairable connection than currently exists. When looking at the partially debonded iterations and the sand iterations, the sand iterations have comparable peak strengths and more sustained strength than the partially debonded iterations. (See Graph 6)



Graph 6: Partially De-Bonded and Sand Iterations Load vs. Slip Comparison

The current codes all consider that the shear stud is fully bonded to the surrounding grout or concrete. AASHTO 2007 section 5.8.4 only has cohesion factors for an as-rolled steel girder connected with headed shear studs. This is the current convention that is used in the industry. When comparing the sand iterations with the fully bonded iterations, the sand iterations, while exhibiting less strength, do maintain the strength longer and have a good peak strength to sustained strength ratio. (See Graph 7)



Graph 7: Fully Bonded and Sand Iterations Load vs. Slip Comparison

6. Conclusions and Future Work

The purpose of this thesis was to analyze one of the connections that needs to be understood more deeply in order for bridges to be more repairable by design. The connection of interest was the shear connection between a steel girder and a concrete bridge deck. The isolation of a headless, debonded shear stud connection gave five results that can be used as a basis for making the traditional shear connection more repairable. These five results also led to five aspects of future work that need to researched for the connection to be more applicable to designers.

6.1 Conclusions

6.1.1 Use of Sand

Placing sand on the top half of the shear stud connection improved residual strength in the connection. The reduction in initial strength from a fully bonded connection fell within the previously studied 22% reduction from the AASHTO 6.10.10 equation. (Issa, 2003) This fact means that the use of sand as a shear transfer material could be utilized in bridge applications where damage loading (earthquakes, etc.) requires some of the shear connectors to have residual strength after the damage loading has occurred.

6.1.2 Debonding and Pull-Off/Separation from the Girder

While all specimens showed evidence of separation of the shear connection from the supporting steel plate, this separation was least noted on the fully bonded studs. These results suggest that the uplift forces seen in a bridge deck do not necessitate the use of as many headed studs as previously thought. Also, the amount of lift-off from the girder remained constant after the initial movement and throughout all of the tests. This consistency suggests again that fewer headed studs may be needed to combat the lift-off.

6.1.3 Debonding and Damage to the Steel Stud

The only steel studs that showed damage were the fully bonded and the fully debonded studs. This result suggests that the level of debonding does not directly correlate with the amount of shear forces directly transferred to the steel stud.

6.1.4 Comparing the Experimental Values with AASHTO Equation 6.10.10 with the 22% Reduction

The iterations of FB_1 and S.5_2 both had minimum load values within 22% of the AASHTO design equation. Additionally, the residual load for both sand iterations was well within the 22% reduction of the AASHTO design equation. These results suggest that the headless stud can be used as a direct replacement for headed studs in the current design equation. Also, the use of alternate materials can be used to transfer the shear between a girder and deck. The shear transfer mechanism is not limited to just grout or concrete.

6.1.5 Comparing the Experimental Results with AASHTO Equation 5.8.4

Both of the fully bonded iterations and S.5_2 had greater minimum strengths than the AASHTO design equation. The other iterations had values proposed for μ and c that ranged from 0.09-0.25 and 0.005-0.025 less than the AASHTO values, respectively. To ensure the safety margins remain intact with the use of headless studs, different design values for c and mu can be codified. This codification would require additional research to validate the required values for mu and c.

While these conclusions, as they are stated, are not capable of being used directly by designers, they do serve as a basis for future researchers. The use of a headless shear stud in the shear connection between a steel girder and the supported deck is a distinct possibility for future bridge designs. The application of a headless shear stud would inherently make the connection between a deck and supporting girder more repairable than the current use of fully bonded, headed studs.

6.2 Future Work

The nature of this experimental program was such that the number of iterations were insufficient in providing any sort of conclusive evidence. The experiment did find some very interesting differences between headed and headless shear studs, between the levels of bonding between grout and shear studs, and between traditional grout and alternate materials used in a shear connection. Any proposed numbers would need to be corroborated by several more iterations.

6.2.1 Sand.

To more fully understand the implications of using sand as a means of transferring shear in the shear stud connection between a bridge deck slab and the supporting girder, additional experimentation is necessary. The level of sand in both iterations used in this experiment was limited to the top half of the shear stud being surrounded by sand. The sand should continue to exhibit the same behavior if it covers half the stud or more. Running an experiment where the level of sand is varied and very closely regulated would aid in understanding the use of sand in the shear stud system.

The reason for only having the sand cover the top half of the shear stud was to ensure that the sand would not leak during the application of a shear load. In this experimental program, the sand did not leak during the application of load when the lower half of the stud was encased in grout. Even the damage caused by the applied load to the grout did not create an opening that allowed any sand to escape. Future experimentation should look at using different materials to restrain the sand, since the shear transfer of the sand itself suggests that the sand will be able to fully transfer the required shear load. The damage of the grout at the base of the shear stud connection is expected due to the large concentration of stresses at the juncture. This damage does not allow for the connection to maintain its strength throughout load application. By replacing the grout with a floating steel plate, a flexible membrane or some means of FRP, the strength of the sand in the shear connection could be isolated. In addition, the strength reduction that occurs when the grout is damaged could also be studied.

Sand was the only alternative material that was considered in this experiment due to the limited number of iterations available. Sand is not the only material that is able to transfer shear. As referenced by Connor, plastic beads transfer shear when applied to a beam application. Using a similar material application could be effective. (Connor, 2003)

6.2.2 Fatigue.

One of the important concerns in the design of bridges is the fatigue loads that the bridge system will see during its service life. Oehlers et al. have extensively studied how the fatigue loading on a shear stud system will affect the overall strength of the system over time. AASHTO has adopted the following equation in Article 6.10.10 of Section 6 to account for the reduction in strength of a shear connection in kips when subject to fatigue loading.

$Z_r = \alpha {d_s}^2 \geq (5.5/2){d_s}^2$ (Eq. 6.10.10.2-1)

with (in kips) α=34.5-4.28*log(N) (Eq. 6.10.10.2-2)

with N=number of cycles and d_s=diameter of shear stud (inches)

This experimental program was aimed at suggesting the ability to use alternative materials and bonding for use in the shear stud system. The basis of the fatigue equations is the static load of each shear stud in the system. To continue the research started in this experiment the application of a fatigue load to sand and debonded shear stud connections should be studied. Since the sand does not become "damaged" during the application of a static load, it is not suspected that it will be damaged during a fatigue load application, which means that the possibility of a higher fatigue load level with a sanded connection is entirely possible.

In addition to the lack of damage, there is no bond to break between the sand and the steel shear stud, which means that the peak load may not be reached at the usual slip when the stud engages. The ductility of the sanded connection under fatigue load could be studied with a thought to earthquake or other catastrophic application. Fatigue loading should be applied to all iterations that prove to have good results after static load tests.

6.2.3 Headed Studs

Currently, headed studs are the most commonly used shear studs. This experiment only looked at smooth headless studs. With the residual strength gain observed during the sanded iterations, the use of a headed stud may provide further interesting results. It would be beneficial to conduct tests in a sanded shear connection with headed studs.

Additionally, this thesis has shown that at least a marginal amount of lift-off of the specimen occurred during each iteration. Therefore, the use of a headed stud cannot be fully taken out of the application to a bridge deck and girder system. However, the varying degree of lift-off indicates that the proportion of headed studs to headless studs may be a variable that can be adjusted to make bridge decks more repairable in the future. Conducting experiments with various combinations of headed studs to headless studs could prove to be further enlightening.

The work of Issa et al. also suggested that the orientation of shear studs set in a cluster could impact the magnitude of shear force that could be resisted. A similar approach can be taken with headed and headless stud combinations. Finding the optimal location of a headed stud placed within a cluster of headless studs or one headed stud on a precast bridge deck panel with all the other clusters filled with headless studs will allow the use of headless studs to be more applicable to bridge designers.

6.2.4 Roughened surface studs.

The interface between the shear stud and the surrounding grout is one where large stresses are transferred and large amounts of damage occur. Consequently, using a roughened interface between the two elements may cause the interface to transfer stresses more effectively with less damage or pullout.

6.2.5 Full scale testing.

This experiment attempted to focus on the steel stud and grout interaction by using a weaker grout and smaller steel stud than would be used in practice. To make the suggestions of this thesis more applicable, the same testing should be conducted on design strength materials.

This thesis has shown that the use of a headless stud in the shear connection between a concrete deck and a supporting steel girder can be done. The headless studs can be used to resist shear in the bridge deck to girder connection. The additional impact that the level of debonding has on the connection is also applicable to making that connection more repairable. The use of alternate materials to transfer shear could also possibly allow the shear connection to be repaired more easily. The results of this thesis can be used as a basis for further investigation into the headless debonded or alternate material shear stud connection. As a starting point in looking at all the connections of a bridge deck to girder individually, this thesis shows that there is much to be gained through a better understanding of this connection and applying it to make the overall system more repairable. The aspects of headless shear studs, in both bonded and debonded states, discussed in the future work section will be applicable to bridge designers desiring to make more repairable bridges in the future.

Appendix A: Photos of Experiment Results

Photos Showing Grout Damage for each Iteration


DB-1
2
3
4


DB-2
2
3
6

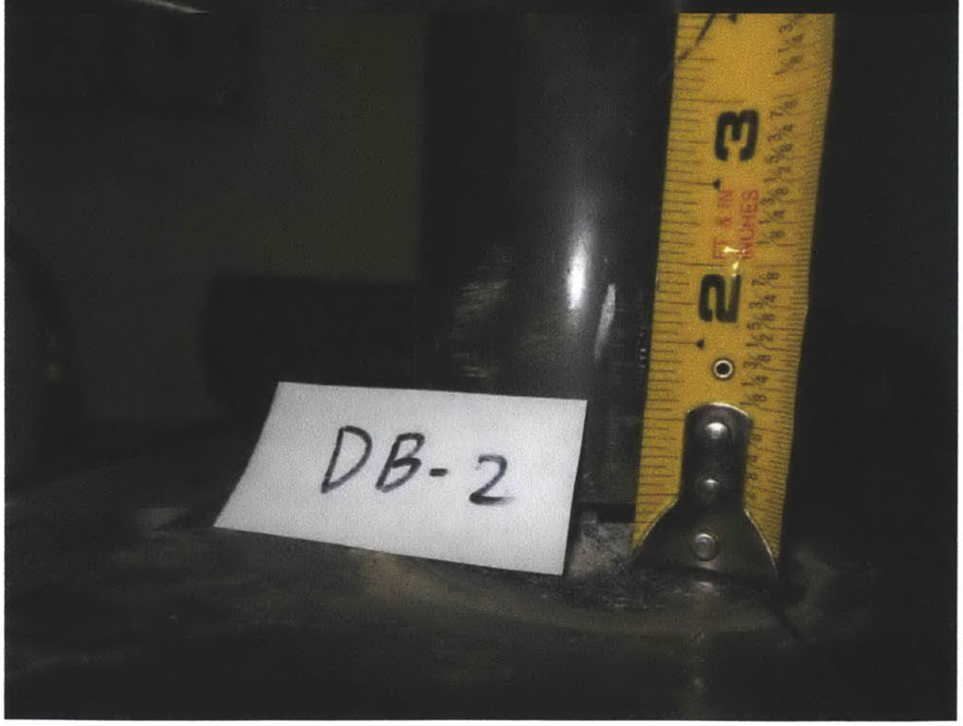

DB5-1


DB5-2


S.5-1
2 FT & IN INCHES 3


S.5-2
2

Pull-off Photos of all specimens that did not fail in Shear



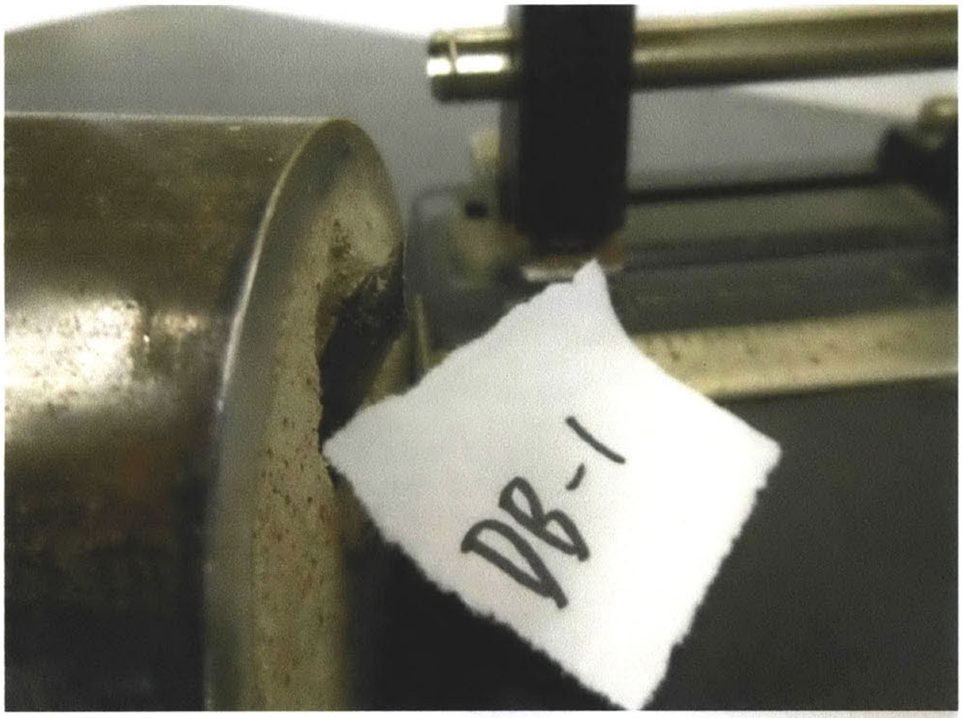

DB.5-1


DB.5-2
FT & IN
INCHES

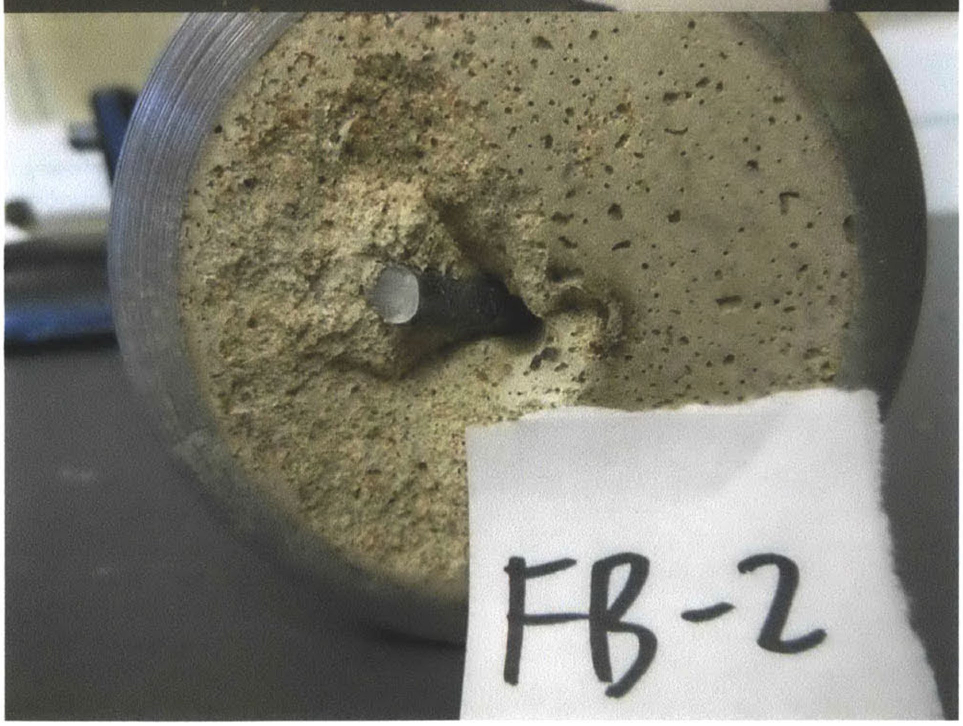

S.5-1
FT & IN
INCHES


S.5-2
FT & IN
INCHES

Steel Stud Damage Photos for all Specimens that exhibited Steel Stud Damage




FB-1


FB-2

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