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I-Girder/Deck Connection for Efficient Deck Replacement

Nebraska Department of Roads (NDOR)
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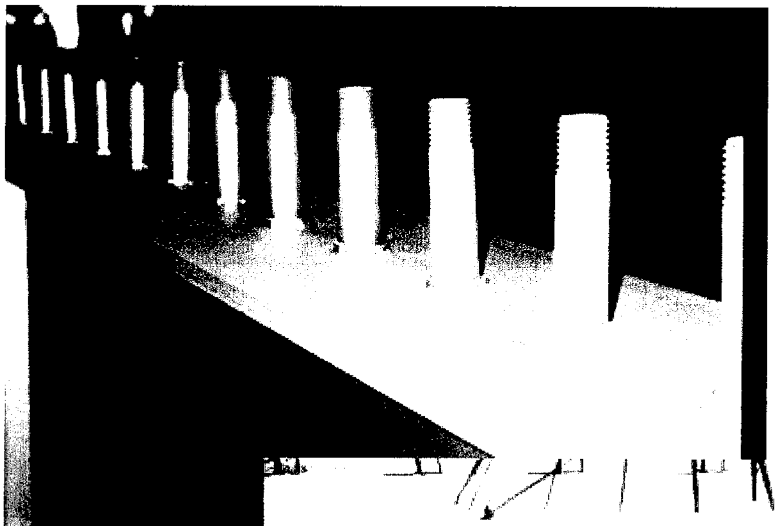

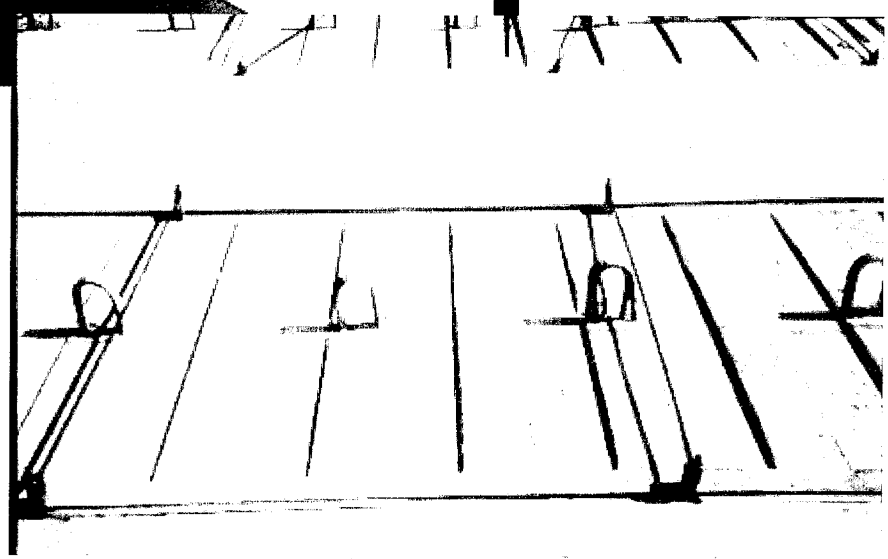

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FINAL REPORT

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13. Abstract

Efficient replacement of bridge decks is becoming increasingly important in high traffic areas due to public intolerance to extended bridge closure. Rehabilitation of deteriorated decks causes public inconvenience, travel delay, and economic hardship. Maintaining traffic flow during bridge deck repair is often difficult and requires extensive planning and coordination. Demolition of bridge deck that is compositely connected with either structural steel I-girders or precast concrete I-girders is one of the major items in deck replacement. This is very time consuming because removing concrete around shear connectors takes time and care to avoid damage to the girders.

Objective of this research is to implement two new connection systems that were developed in the NCHRP project 12-41, one system for steel girder/concrete deck connection and the other for concrete girder/concrete deck connection.

For steel girder/concrete deck connection, a new large 31.8 mm (1¼ in.) diameter shear stud is used to replace popular 19.1 mm (¾ in.) and 22.2 mm (7/8 in.) shear studs. Using the 31.8 mm (1¼ in.) studs result in higher construction speed and enhancement of deck removal.

For concrete girders, a new debonded shear key system is used to replace the conventional roughened interface system. Shear friction theory is adopted to develop a design procedure for this new connection. Extensive tests and field implementation showed that new shear key system has comparable structural behavior with conventional roughened interface system.

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ASBSTRACT

Efficient replacement of bridge decks is becoming increasingly important in high traffic areas due to public intolerance to extended bridge closure. Demolition of bridge deck that is compositely connected with either structural steel I-girders or precast concrete I-girders is one of the major items in deck replacement. This is very time consuming because removing concrete around shear connectors takes time and care to avoid damage to the girders.

Objective of this research is to implement two new connection systems that were developed under the NCHRP project 12-41, one system for steel girder/concrete deck connection and the other for concrete girder/concrete deck connection.

For steel girder/concrete deck connection, a new large 1¼ in. (31.8 mm) diameter shear stud is used to replace popular ¾ in. (19 mm) and 7/8 in. (22.2 mm) shear studs. Using the 1¼ in. (31.8 mm) studs result in higher construction speed and enhancement of deck removal.

For concrete girders, a new debonded shear key system is used to replace the conventional roughened interface system. Shear friction theory is adopted to develop a design procedure for this new connection. Extensive tests and field implementation showed that new shear key system has comparable structural behavior with conventional roughened interface system.

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EXECUTIVE SUMMARY

Project Objectives

The main objectives of this research are:

1. Implement into bridges in Nebraska the two new connection systems for composite bridges that were developed under the National Cooperative Highway Research Program (NCHRP), Project 12-41. One system for steel girder/concrete deck bridges and the other for concrete girder/concrete deck bridges.
2. Develop design criteria and specifications for the new systems
3. Monitor the structural performance of the new systems and compare it with that of the conventional systems.
4. Develop cost analysis of the new systems based on the experience gained from the field demonstration projects

Background

Efficient replacement of bridge decks is becoming increasingly important in high traffic areas due to public intolerance to extended bridge closure. Rehabilitation of deteriorated decks causes public inconvenience, travel delay, and economic hardship. Maintaining traffic flow during bridge deck repair is often difficult and requires extensive planning and coordination. A project entitled "Rapid Replacement of Bridge Decks" sponsored under the National Cooperative Highway Research Program (NCHRP), Project 12-41, Report #407, was performed by UNL to identify solutions to these problems.

Demolition of a bridge deck that is compositely connected with either structural steel I-girders or precast concrete I-girders is one of the major items in deck replacement.

This is very time consuming because removing concrete around shear connectors takes time and care to avoid damage to the girders, as well as creation of excessive noise and debris. This is especially true with modern precast concrete I-girders with wide thin top flanges, such as the NU I-girder. This delay can be reduced by constructing bridges with connections for composite action that are relatively easy to remove.

Two new connection systems were developed under the NCHRP project, one for concrete girder/concrete deck connection and the other for steel girder/concrete deck connection.

For concrete girders, a debonded shear key system was developed. Shear friction theory was adopted to develop a design procedure for this new connection. Extensive tests were performed in the laboratory on push-off specimens and lab size beam specimens. Full-scale tests were also performed at the testing facility at Wilson Concrete Co., La Platt, NE. All test results confirmed the expected theoretical behavior and that the system was ready for implementation.

For steel girder/concrete deck connection, a new large 31.8 mm (1¼ in.) diameter shear stud was developed to replace the popular 19 mm (¾ in.) and 22.2 mm (7/8 in.) shear studs. The 19 mm (¾ in.) and 22.2 mm (7/8 in.) shear studs are often provided in two or three studs per row spaced in the range of 127 to 457 mm (5 to 18 in.). A large number of small studs, results in increased concrete removal time, and higher probability of stud damage and of girder top flange damage. The new 31.8 mm (1¼ in.) stud, which provides approximately twice the capacity of 22.2 mm (7/8 in.) stud, would allow positioning the studs in a single row over the girder web. This arrangement would greatly reduce the amount of jack hammering since saw cutting the old concrete could be very close to the girder centerline. Also, alternating headed and headless studs was found by UNL researchers to be all that is needed for anchorage to the concrete deck. This should further facilitate deck removal.

The limited fatigue testing performed on 31.8 mm (1¼ in.) studs shows that these studs have higher fatigue resistance than the conventional 22.2 mm (7/8 in.) studs. UNL researchers are not quite satisfied with the number of tests done on this issue, and are not ready to make final recommendations until additional tests are undertaken. A full-scale test was performed on a 12.2-m (40-ft) long steel girder with the 31.8 mm (1¼ in.) diameter shear studs. The girder was designed for full HS-25 truck loading, without giving the larger studs any credit for the higher fatigue resistance observed in the above-mentioned tests. The test results confirmed the outstanding performance observed in small-scale push-off specimen testing.

Research Approach and Methods

The following plan was used to achieve the research objectives:

(I) For the Debonded Shear Key System for Composite Concrete Girders

1. Finalize the details of the debonded shear key system.
2. Develop the design criteria and specifications for the new system.
3. Assign a twin bridge project where the concrete NU girder of one bridge is designed with the new system and the other bridge is designed with the conventional system.
4. Document the effect of the new shear key system on the production of the NU girders and on the bridge construction.

5. Monitor long-term structural behavior of the bridge built with the new system to determine the relative performance of the new system.
6. Develop a cost analysis for the new system based on experience gained from the demonstration project.

<u>(II) For the 1¼ in. (31.8 mm) Large Studs for Steel Composite Girders</u>

1. Verify the fatigue capacity of the 31.8 mm (1¼ in.) and 22.2 mm (7/8 in.) shear studs by developing the α factor of the stud fatigue capacity equation. A large number of push-off specimens will be tested in fatigue for different stress ranges. The specimens will be grouped in two groups. One group will have 31.8 mm (1¼ in.) shear studs and the other will have 22.2 mm (7/8 in.) shear studs. Each group will divided into many sub groups for different stress ranges. Statistical analysis will be conducted to determine the α-values in relation to the number of cycles.
2. Compare the results of the fatigue investigation with the AASHTO Standard and Load and Resistance Factor Design (LRFD) Specifications and propose changes to the specifications, if needed.
3. Develop the design criteria and specifications of the quality control test for the new 31.8 mm (1¼ in.) studs.
4. Design a steel girder bridge with the new stud connection details. The selected bridge should have at least two spans. One span would be designed with the conventional studs and the other span with the new studs.
5. Document welding process of the 31.8 mm (1¼ in.) studs on the bridge steel girders.

6. Monitor the long-term structural behavior of the bridge deck built with the new system to determine the relative performance of the new system.
7. Develop a cost analysis for the new system based on experience gained from the demonstration project.

TABLE OF CONTENTS

CHAPTER 3: DEBONDED SHEAR KEY SYSTEM FOR CONCRETE COMPOSITE CONSTRUCTION

LIST OF FIGURES

CHAPTER 1: INTRODUCTION

Nil

CHAPTER 2: LARGE STUDS FOR STEEL COMPOSITE GIRDERS

CHAPTER 4: CONCLUSIONS AND RECOMMENDATIONS

LIST OF TABLES

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

Efficient replacement of bridge decks is becoming increasingly important in high traffic areas due to public intolerance to extended bridge closure. Rehabilitation of deteriorated decks causes public inconvenience, travel delay, and economic hardship. Maintaining traffic flow during bridge deck repair is often difficult and requires extensive planning and coordination. A project entitled "Rapid Replacement of Bridge Decks" sponsored under the National Cooperative Highway Research Program (NCHRP), Report #407 [Tadros & Baishya (1998)] was performed by UNL to identify solutions to these problems.

Demolition of a bridge deck, that is compositely connected with either structural steel I-girders or precast concrete I-girders, is one of the major items in deck replacement. This is very time consuming because removing concrete around shear connectors takes time and care to avoid damage to the girders, as well as creation of excessive noise and debris. This is especially true with modern precast concrete I-girders with wide thin top flanges, such as the NU I-girder. This delay can be reduced by constructing bridges with connections for composite action that are relatively easy to remove. This is analogous to having parts of a model car that are snapped or bolted together, rather than glued or welded.

Two new connection systems were developed under NCHRP project 12-41 [Tadros & Baishya (1998)], one for steel girder/concrete deck connection and the other for concrete girder/concrete deck connection.

For steel girder/concrete deck connection, a new large 31.8 mm (1¼ in.) diameter shear stud was developed to replace the popular 19.1 mm (¾ in.) and 22.2 mm (7/8 in.) shear studs. The 19.1 mm (¾ in.) and 22.2 mm (7/8 in.) shear studs are often provided in two or three studs per row spaced in the range of 101.6 to 457.2 mm (4 to 18 in.). A large number of small studs, results in increased concrete removal time, and higher probability of stud damage and of girder top flange damage. The new 31.8 mm (1¼ in.) stud, which provides approximately twice the capacity of the 22.2 mm (7/8 in.) stud, would allow positioning in a single row over the girder web. This arrangement would greatly reduce the amount of jack hammering since saw cutting the old concrete could be very close to the girder centerline. Also, alternating headed and headless studs was found by UNL researchers to be all that was needed for anchorage to the concrete deck. This should further facilitate deck removal.

The limited fatigue testing performed on 31.8 mm (1¼ in.) studs [Kakish (1997), Badie et al. (2000)] shows that these studs have higher fatigue resistance than the conventional 22.2 mm (7/8 in.) studs. UNL researchers are not quite satisfied with the number of tests done on this issue, and are not ready to make final recommendations before additional tests are undertaken. A full-scale test was performed on a 12.2-m (40-ft) long steel girder with the 31.8 mm (1¼ in.) diameter shear studs. The girder was designed for full HS-25 truck loading, without giving the larger studs any credit for the higher fatigue resistance observed in the above-mentioned tests. The test results confirmed the outstanding performance observed in a small-scale push-off specimen testing.

For concrete girders, a debonded shear key system was developed. Shear friction theory was adopted to develop a design procedure for this new connection. Extensive tests were performed in the laboratory on push-off specimens and lab size beam specimens. Full-scale tests were also performed at the testing facility at Wilson Concrete Co., La Platt, Nebraska. All test results [Kamel (1996)] confirmed the expected theoretical behavior and that the system is ready for implementation.

1.2 RESEARCH OBJECTIVES

The main objective of this research is to implement the new connection systems developed in the NCHRP research project [Tadros & Baishya (1998)] in bridges in Nebraska and investigate their structural performance.

1.2.1 Large Studs [31.8 mm (1¼ in.)] for Steel Composite Girders

The implementation plan for the 31.8 mm (1¼ in.) large studs for steel composite girders is as follows:

1. Verify the fatigue capacity of the 31.8 mm (1¼ in.) and 22.2 mm (7/8 in.) shear studs by developing the α factor of the stud fatigue capacity equation,

   $$Z_r = \alpha d^2 \qquad \text{Equation 1.1}$$

   Where:

   Zr = the shear capacity of the stud

   d = stud diameter

   A large number of push off specimens will be tested in fatigue for different stress ranges. The specimens will be grouped in two groups. One group will have 31.8 mm (1¼ in.) shear studs and the other will have 22.2 mm (7/8 in.) shear studs. Each group will divided into many sub groups for different stress ranges. Statistical analysis will be conducted to determine the α values in relation to the number of cycles.

2. Compare the results of the fatigue investigation with the AASHTO Standard (1996) and LRFD (1998) Specifications and propose changes to the specifications if needed.
3. Document the design criteria and specifications of the quality control test for the new 31.8 mm (1¼ in.) studs.
4. Design a steel girder bridge with the new stud connection details. The selected bridge should have at least two spans. One span would be designed with the conventional studs and the other span with the new studs.
5. Document welding process of the 31.8 mm (1¼ in.) studs on the bridge steel girders.
6. Monitor long-term structural behavior of the bridge deck built with the new system to determine the relative performance of the new system.
7. Prepare a cost analysis for the new system based on the demonstration project.

1.2.2 Debonded Shear Key System for Composite Concrete Girders

The implementation plan for the debonded shear key system for composite concrete girders is as follows:

1. Finalize the details of the debonded shear key system.
2. Document the design criteria and specifications for the new system.
3. Assign a twin bridge project where the concrete NU girder of one bridge is designed with the new system and the other bridge is designed with the conventional system.
4. Document production process of the NU girders made with the new shear key system.
5. Document the effect of the new shear key system on the production of the NU girders and on the bridge construction.

6. Monitor long-term structural behavior of the bridge built with the new system to determine the relative performance of the new system.
7. Prepare a cost analysis for the new system based on the demonstration project.

CHAPTER 2

LARGE STUDS FOR STEEL COMPOSITE GIRDERS

2.1 INTRODUCTION AND BACKGROUND

The 31.8 mm (1¼ in.) stud was developed at the University of Nebraska under a project titled "Rapid Replacement of Bridge Decks" [Tadros and Baishya (1998)] funded by the National Cooperative Highway Research Program (NCHRP), Project 12-41. Tests conducted in this project [Kakish (1997), Tadros and Baishya (1998)] showed that the 31.8 mm (1¼ in.) diameter shear studs are an efficient replacement for the conventional 22.2 mm (7/8 in.) diameter studs. With the use of the 31.8 mm (1¼ in.) studs, fewer number and/or wider spacing of studs can be used, which results in saving in construction time and ease of deck replacement in the future. The NCHRP project concentrated on the development of the 31.8 mm (1¼ in.) stud, the quality control test, and checking ultimate strength through push-off specimens and full-scale beam testing.

2.1.1 Results Obtained from the NCHRP Project 12-41

Results obtained from the NCHRP Project 12-41 [Tadros and Baishya (1998)], regarding the development of the 31.8 mm (1¼ in.) stud, can be summarized as follows:

(a) Stud material: The Standard cold drawn, Society of Automotive Engineers (SAE), 1018 steel can be used to produce the 31.8 mm (1¼ in.) shear studs instead of SAE 1008, which is currently used to produce the 22.2 mm (7/8 in.) studs. This decision was made due to the local shortage of the SAE 1008 rods with 31.8 mm (1¼ in.) diameter and after a study [Kakish (1997), Tadros and Baishya

(1998)] conducted in cooperation with a stud manufacturer to find a local available source. SAE 1018 steel has higher carbon content and lower ductility than SAE 1008 steel. **Table 2.1** gives a comparison of the mechanical properties between SAE 1008 and SAE 1018.

(b) Stud dimensions: The 31.8 mm (1¼ in.) stud has steeper chamfer than that used with the 22.2 mm (7/8 in.) studs to facilitate the stud welding. **Figure 2-1** gives the dimensions of the new stud. Also, the amount of flux used is triple of that used with the 22.2 mm (7/8 in.) studs. The proposed stud can be produced as headed or headless stud. The headed stud is produced by providing threads in the top 38.1 mm (1½ in.) of the stud height and a nut of 31.8 mm (1¼ in.) internal diameter. Stud-gun, that is utilizing arc welding and is currently used for welding smaller studs, can be used to weld the studs to the steel girder. However, relatively high Amperage (2,500 to 2,900 Amp.) is needed to weld the new stud.

(c) Quality control: Welding quality can be checked by shearing off two adjacent studs using a small hydraulic jack and pump. This test will be discussed in detail later in this chapter.

(d) Stud ultimate capacity:

- AASHTO LRFD Equation 6.10.7.4.4c-1 can be used to predict the 31.8 mm (1¼ in.) stud ultimate capacity.

$Q_n = 0.5\, A_{SC} \sqrt{f'_c E_c} \leq A_{SC} F_u$ LRFD EQ. 6.10.7.4.4c-1, SI Units

$Q_n = 0.5\, A_{SC} \sqrt{f'_c E_c} \leq A_{SC} F_u$ LRFD EQ. 6.10.7.4.4c-1, English Units

Where: A_{sc} = stud cross sectional area mm^2 (in^2)

f'_c = specified 28-day compressive strength of concrete MPa (ksi)

E_c = modulus of elasticity of concrete MPa (ksi)

F_u = specified minimum tensile strength of a stud MPa (ksi)

Q_n = nominal stud shear resistance N (kip)

Since the new stud has almost twice the cross sectional area of the 22.2 mm (7/8 in.) stud, one 31.8 mm (1¼ in.) stud can replace two 22.2 mm (7/8 in.) studs. This finding was confirmed by testing a large number of push-off specimens.

- The center-to-center pitch of the 31.8 mm (1¼ in.) studs shall not be less than 152.4 mm (6 in.).
- It is recommended to use continuous top and bottom reinforcement transversely to the girder lines to provide adequate confinement of the concrete around the stud and to achieve the ultimate stud capacity given by the LRFD Specifications. This issue is recognized by the empirical deck design of the AASHTO LRFD Specifications (1998), which mandates the use of continuous top and bottom reinforcement over girder lines. Tests conducted on a limited number of push-off specimens where top and bottom transverse reinforcement were fully developed showed that the 31.8 mm (1¼ in.) stud material could be fully developed, i.e. the stud ultimate capacity is controlled by its ultimate tensile strength.
- The specimens that were made with 31.8 mm (1¼ in.) studs showed about 30 percent less slippage at failure than the specimens made with the 22.2 mm (7/8 in.) studs.

- Cyclic loading has no detrimental effect on the ultimate capacity of the 31.8 mm (1¼ in.) stud.
- Residual stresses due to replacement of the 22.2 mm (7/8 in.) studs with 31.8 mm (1¼ in.) studs appeared to have no detrimental effect on the capacity of the 31.8 mm (1¼ in.) stud.
- Replacing 50 percent of the headed studs with headless studs resulted in a reduction of the stud capacity by 17 percent. It is difficult to estimate the reduction when headed studs are fully replaced with headless studs. Use of headless studs may appear to be disadvantageous due to the anticipated relative large number of studs needed for full composite action. However, ease of long-term deck removal and replacing may more compensate for the increased number of headless studs used. Further study is needed in this area.

(e) Full-scale beam testing: Linear stress distribution over the beam height due to live load was observed before and after applying fatigue load. Fatigue test showed no loss of composite action between the concrete deck and the steel beam or distress in the concrete deck due to the use of 31.8 mm (1¼ in.) studs.

(f) Stud fatigue capacity: Fatigue testing on a limited number of the 31.8 mm (1¼ in.) stud push-off specimens [Kakish (1997), Tadros and Baishya (1998)] showed that the 31.8 mm (1¼ in.) studs have higher fatigue capacity than that of the 22.2 mm (7/8 in.) and the 19 mm (¾ in.) studs, as shown in **Figure 2.2**. Based on this figure, Kakish (1997) recommended using higher values of the coefficient α than those given by the AASHTO Standard and LRFD Specifications to determine the allowable range of horizontal shear that can be carried by the 31.8 mm (1¼ in.) studs. Listed in **Table 2.2** are the α-values given by the AASTHO Standard and LRFD Specifications as well as the modified values that were recommended by Kakish (1997). Kakish (1997) concluded that: (1) additional tests are needed to get more accurate values of the constant α for the 31.8 mm (1¼ in.) studs, and (2) it is very conservative to use the α-values given by the AASHTO Standard Specifications [AASHTO Standards (1996)] and the AASHTO LRFD Specifications [AASHTO LRFD (1998)] to calculate the allowable range of horizontal shear force for the 31.8 mm (1¼ in.) and the 22.2 mm (7/8 in.) studs.

2.2 RESEARCH OBJECTIVES

Objectives of this part of the research project were to:

(a) Finalize the quality control test procedure for the 31.8 mm (1¼ in.) studs.

(b) Investigate the fatigue strength of the 31.8 mm (1¼ in.) studs: Determine the α-values in relation to the number of cycles for the 31.8 mm (1¼ in.) studs and compare them to those of the 22.2 mm (7/8 in.) studs given in the AASHTO Specifications. Push-off specimens are tested in fatigue for different stress ranges. Two groups of specimens are used. One group has 31.8 mm (1¼ in.) stud specimens and the other has 22.2 mm (7/8 in.) stud specimens. Each group is divided into many sub-groups for different stress ranges.

(c) Implement the 31.8 mm (1¼ in.) studs on a demonstration bridge and document their installation.

(d) Monitor the structural behavior of the demonstration bridge.

(e) Prepare an economy analysis of using the 31.8 mm (1¼ in.) studs.

2.3 QUALITY CONTROL TEST

Most state agency specifications require that to inspect the quality of the stud welding, the stud should be bent 45 degrees without failure. Although this procedure is considered an easy and inexpensive solution for the 22.2 mm (7/8 in.) diameter studs, it is not convenient for the 31.8 mm (1¼ in.) diameter studs because of the large force required to bend the stud. Therefore, a portable hydraulic jacking system that could be used in the field or in the shop was developed [Kakish (1997), and Tadros and Baishya (1998)].

The structural model of the test setup is shown in **Figure 2.3**. It consists of two collars placed around two adjacent studs, a small hydraulic jack, and a top tie. The collar consists of two steel blocks tied together with four screws. By tightening the four screws, the collar is placed in contact with the 31.8 mm (1¼ in.) stud. The base of the collar is recessed to accommodate the weld at the stud base. A small hydraulic jack, of 444.8 kN (100-kip) capacity, is placed between the collars to provide lateral shearing force at the stud base. The top tie, which consists of two hooks and a turnbuckle, is used to protect the studs from bending and to provide safety for the technicians during the test. The device was successfully used to test studs used in the experimental program as well as the studs of the implementation bridge. **Figure 2.4** shows the test setup during testing.

The quality control test is conducted by applying a horizontal force that would cause an axial tension failure in the stud. This force can be calculated by analyzing the studs with the top tie as a closed frame action, where the studs are fixed at their base and hinged at the top. Loads applied by the hydraulic jack on the studs are: (1) a distributed load over the collar height, which is in contact with the stud and (2) a moment due to misalignment of the hydraulic jack and the collar. Frame analysis of the model shows that the maximum bending moment occurred at the stud base and equals 0.074F kip-ft, where F is the total force applied by the hydraulic jack, and the maximum shear force also occurred at the stud base and equals 0.791F. Since maximum shear stress occurs at the centroid of the cross section where the bending stress is zero; principle tensile stress, σ, equals shear stress, ν. Thus,

$$\sigma = \nu = \frac{16}{3\pi d_s^2} V \qquad \text{Equation 2.1}$$

Where:

ν = maximum shear stress (ksi)

V = applied shear force = 0.791F kips

d_s = stud diameter = 1.25 in.

Substituting the principle stress with the yield stress of the stud, f_y, would result in the following formula.

$$F = 1.164 f_y \qquad \text{Equation 2.2}$$

In order to protect the stud from damage during the quality control test, an average factor of safety of two may be applied to Equation 2.2. For example, if the specified yield strength of the stud material = 372 MPa (54 ksi) as given in Table 2.1, and an average factor of safety of two is used. Thus, safe testing load, F = (1.164)(54)/2 = 31.4 kips.

2.4 FATIGUE INVESTIGATION

In order to establish the relation between the number of cycles and the stress range, two parameters that have great impact on the allowable stud shear capacity were investigated. These parameters were (1) stress range and (2) minimum stress applied to the stud. These parameters are discussed in the following sections.

2.4.1 Stress Range, S_r

In a parametric study conducted by Kakish (1997), simple span bridges of 9.1, 27.4, and 36.6 m (30, 90, and 120 ft) span were analyzed. Two-girder spacing of 1.8 and 3.7 m (6 and 12 ft) were considered for each span. The bridges were analyzed under HS25 Truck Load or the equivalent live lane load, according to the AASHTO Standard Specifications [AASHTO Standard, 16 Edition, 1998]. Maximum stress range found by this study was 3.3 kips/in. Thus, if 6 in. stud spacing is considered, the stress range on the 31.8 mm (1¼ in.) stud = (3.3 kips/in.)(6 in.)/($\frac{\pi}{4}$x1.25^2 in^2) = 16.13 ksi.

Literature review of tests conducted on stud shear connectors [Thurlimann (1958), Thurlimann (1959), King et al. (1965), Toprac (1965), and Slutter and Fisher (1966)] showed that stress range as low as 55.16 MPa (8 ksi) and as high as 165.48 MPa (24 ksi) were used, as shown in **Table 2.3**.

Based on these studies, the research team decided to use a stress range between 68.95 to 172.38 MPa (10 and 25 ksi). This range covered all of the stress ranges used before in the literature and at the same time it had a mean value of 120.66 MPa (17.5 ksi) that was very close to that calculated by the parametric study conducted by Kakish (1997).

2.4.2 Minimum Stress

The minimum horizontal stress on a stud is caused by the superimposed dead loads, i.e. barrier and wearing surface weight. In an attempt to get a feel for the minimum stress in bridges, a bridge with the following characteristics was considered:

(a) Total width = 9.8 m (32 ft), three girders spaced at 3.66 m (12 ft) with two overhangs each of 1.2 m (4 ft) long,

(b) Simple span of 36.6 m (120 ft)

(c) Steel plate girder: web 1727.2x19.1 mm (68x0.75 in.), top flange 406.4x25.4 mm (16x1 in.), bottom flange 457.2x50.4 mm (18x2 in.)

(d) Deck slab with 203.2 mm (8 in.) thickness.

(e) New Jersey barriers of 8.8 kN/m/side (0.6 kip/ft/side)

(f) Concrete wearing surface of 50.4 mm (2 in.) thick

Thus, maximum shear force per girder due to superimposed dead load equals

$$[(\frac{2x12}{12}x0.15)+(\frac{2x0.6}{3})](\frac{120}{2}) = 42 \text{ kips}$$

$$\text{Shear flow at stud level} = \frac{VQ}{I} = \frac{42x2043}{174{,}529} = 0.49 \text{ kip/in.}$$

With 6 in. stud spacing, minimum stress equals

$$(0.49 \text{ kips/in.})(6 \text{ in.})/(\frac{\pi}{4}x1.25^2 \text{ in}^2) = 2.4 \text{ ksi}$$

Review of the literature [Thurlimann (1958), Thurlimann (1959), King et al. (1965), Toprac (1965), and Slutter and Fisher (1966)] showed that a minimum stress ranges between 13.79 to 27.58 MPa (2 and 4 ksi) was used in fatigue testing. Based on these studies, the research team decided to use a minimum stress of 34.48 MPa (5 ksi) as a conservative figure.

2.4.3 Testing Program

Table 2.4 gives the testing plan of the 31.8 mm (1¼ in.) and 22.2 mm (7/8 in.) diameter stud specimens. Twenty-five push-off specimens (fourteen specimens with 31.8 mm (1¼ in.) studs and eleven specimens with 22.2 mm (7/8 in.) studs) were tested. A minimum stress of 34.48 MPa (5 ksi) was used for all specimens. The stress range varied from 68.95 to 172.38 MPa (10 to 25 ksi). The design specified concrete strength was chosen to be 27.58 MPa (4 ksi) after 28 days in order to investigate the effect of the fatigue load on low strength concrete around the stud. In addition, the 27.58-MPa (4-ksi) concrete strength represents the lower bound of concrete strength currently used for deck slab in bridges in Nebraska and in most of the states.

2.4.4 Test Specimens

Figure 2.5 shows the details of the test specimen. An L-shape specimen was used. Each specimen had two studs welded to a 12.7 mm (½ in.) thick steel plate. Spacing between the studs was 152.4 mm (6 in.). To provide anchorage for the transverse reinforcement in the specimen, 19.1 mm (¾ in.) diameter transverse threaded bars were used. These bars were mechanically anchored at the sides of the specimen by using steel plates and nuts. Before running the test, the nuts were tightened using a crescent wrench.

A local company, Tri Sales Associates, Omaha, NE, welded the 22.2 mm (7/8 in.) and the 31.8 mm (1¼ in.) studs to the steel plates. A tri-leg support was used during welding of the 31.8 mm (1¼ in.) studs on the first couple of steel plates to adjust the verticality of the studs as shown in **Figure 2.6a**. However, after the technician got enough confidence in the welding process, he shot the rest of the 31.8 mm (1¼ in.) studs without using the tri-leg support, as shown in **Figure 2.6b**. A power supply of 2,900 Amps was used in welding the 31.8 mm (1¼ in.) studs. **Figures 2.7a and 2.7b** show the welding quality of the 31.8 mm (1¼ in.) and the 22.2 mm (7/8 in.) studs, where perfect welding around the stud was achieved.

Wood was used to form the specimens and expanded polystyrene was used to form the blockouts as shown in **Figures 2.8a and 2.8b**. Specimens were cast in two groups, the first group for the 31.8 mm (1¼ in.) stud specimens and the second group for the 2.22 mm (7/8 in.) stud specimens. Concrete mix of a specified compressive strength of 20.69 MPa (3 ksi) at 28 days was ordered from Ready Mix Co., Omaha, Nebraska, for both groups. Components of the concrete mix were as follows:

Sand	950 lb/cu yd
Crushed lime stone	2245 lb/cu yd
Cement	517 lb/cu yd
Water	144 gl/cu yd (266 lb/cu yd)
Unit weight	147 lb/cu ft
Water Cement ratio	0.52

Concrete cylinders cast and cured beside the specimen were prepared to monitor the compressive strength gain. **Figure 2.9** gives the compressive strength versus time. Although the specified concrete strength after 28 days was 20.69 MPa (3 ksi), the concrete mix reached a compressive strength of 35.85 MPa (5.2 ksi) for the 31.8 mm (1¼ in.) specimens and 30.34 MPa (4.4 ksi) for the 22.2 mm (7/8 in.) specimens after 28 days.

2.4.5 Test Setup

The specimen was attached to an L-shape steel beam by ten 22.2 mm (7/8 in.) diameter bolts. Dimensions of the steel beam are given in **Figure 2.10**. A self-equilibrium frame was used to test the specimen as shown in **Figure 2.11,** where the cyclic (fatigue) load is applied through a 489.3-kN (110-kip) axial load actuator. The centerline of the actuator was at the same elevation as the stud base. Thus, only shear force was applied to the studs. No over turning moment existed. **Figure 2.12** shows the testing frame during testing one of the specimens.

Cyclic load testing was carried out at fixed frequency for all of the specimens, which was 2 cycles/sec. This frequency was chosen to maintain stability of the testing frame. Maximum and minimum loads applied to the specimen were determined based on stud size, minimum stress, and stress range. For example, for the specimen designated as "LS-5-15A", 31.8 mm (1¼ in.) diameter stud, 34.48 MPa (5 ksi) minimum stress, and 103.43 MPa (15 ksi) stress range, minimum load = $(2\text{x}\frac{\pi}{4}\text{x}1.25^2\ \text{in}^2)(5\ \text{ksi}) = 12.3$ kips,

maximum load = $(2\text{x}\frac{\pi}{4}\text{x}1.25^2\ \text{in}^2)(5+15\ \text{ksi}) = 49.1$ kips.

Linear Voltage Displacement Transducers (LVDT) device was attached between the specimen and the L-steel frame to measure the slippage of the specimen. Load and slippage were measured using a Data Acquisition System. First, a monotonic load with the maximum load was applied gradually to the specimen and then the load was released to zero. This was done to eliminate any residual slippage that might occur due to specimen fabrication. The same monotonic load was applied again and slippage and applied load were recorded. Then, the cyclic load was applied for 2,000,000 cycles or up to failure of the specimen, whichever came first, at a frequency of 2 cycles per second. After the fatigue load was applied, the maximum monotonic load was applied and slippage and load were recorded again. A plot of slippage against applied load was drawn for two cases. The first case before applying the cyclic (fatigue) load and the second case after applying the fatigue load. This was done to observe the effect of the cyclic load on the slippage

2.4.6 Test Results and Discussion

Table 2.5 summarizes the test results, where the maximum number of cycles attained and the type of failure are given. Three modes of failure were recorded. These were: (1) stud-failure (15 specimens), (2) base-plate failure (2 specimens), and (3) concrete failure (3 specimens).

Figures 2.13a and 2.13b show the stud failure mode of the 31.8 mm (1¼ in.) studs and the 22.2 mm (7/8 in.) studs, where the failure mode for both studs was almost identical. The stud-failure was initiated at the stud weld and penetrated into the base plate causing a concave depression in the base plate. The concave depression of the

31.8 mm (1¼ in.) studs was deeper than that of the 22.2 mm (7/8 in.) studs. This occurred because of the high energy and heat released during welding of the 31.8 mm (1¼ in.) studs, which resulted in a deeper melted depth of the steel base compared to that when welding a 22.2 mm (7/8 in.) stud. The concave depression mode observed in the current research project is consistent with that observed by Slutter and Fisher (1966) and Ollgaard et al. (1971). The stud failure mode occurred at a relatively high stress range, between 124.11 and 172.38 MPa (18 and 25 ksi) for the 31.8 mm (1¼ in.) stud specimens and between 103.43 and 172.38 MPa (15 and 25 ksi) for the 22.2 mm (7/8 in.) stud specimens.

Figure 2.14 shows the base-plate failure mode, where a fatigue tear-off failure occurred in the steel plate in the vicinity of the weld. This mode of failure was observed only in two 31.8 mm (1¼ in.) stud specimens, which were tested at relatively moderate stress range, between 110.32 and 117.22 MPa (16 and 17 ksi). The base-plate failure mode gave the authors a chance to measure the depth of the steel plate that was melted under the stud base during the welding process. It was clear, as shown in **Figure 2.14**, that the melted depth was between 11.1 and 12.7 mm (7/16 and 1/2 in.). To avoid this type of failure in bridges, it is recommended that the 31.8 mm (1¼ in.) studs be welded on steel plates that have thickness greater than 12.7 mm (½ in.). This recommendation is consistent with most of the state agencies' policies, which mandate that the top flange minimum thickness should be 19.1 mm (¾ in.) or greater.

Figure 2.15 shows the concrete failure mode. This mode of failure occurred only in two specimens out of the twenty-five specimens. It was initiated at the free end of the specimen (the unloaded end). The authors believe that it occurred because the side plates, which should provide compression stresses to confine the concrete around the studs, were not in full contact with the concrete surface due to some surface roughness of the specimen sides. In bridges, this mode of failure is not expected to occur because the studs are fully confined within the surrounding concrete of the deck slab, and the deck slab is under flexural compression stresses.

At relatively low stress range, less than 103.43 MPa (15 ksi) for the 31.8 mm (1¼ in.) studs and less than 96.53 MPa (14 ksi) for the 22.2 mm (7/8 in.) studs, no failure was achieved.

Figures 2.16a and 2.16b show the slippage-load relationship before and after applying the fatigue load for the 31.8 mm (1¼ in.) studs, Specimen LS-5-14, and 22.2 mm (7/8 in.) studs, Specimen SS-5-14. Each figure gives the relationship after one cycle and after 2,000,000 cycles. Both specimens showed almost the same amount of increase in slippage. Maximum recorded slippage after 2,000,000 cycles for both specimens was about 0.53 mm (0.021 in.), which is lower than that achieved in the ultimate test. The authors believe that this was a result of eliminating eccentricity of the test setup and having better concrete confinement around the studs. The 31.8 mm (1¼ in.) stud specimens showed a slippage increase of about 23 percent, while the 22.2 mm (7/8 in.) stud specimens showed an increase of about 17 percent. All of the 31.8 mm (1¼ in.) and 22.2 mm (7/8 in.) stud specimens that had stud failure showed almost the same slippage-load relationship.

Figures 2.17a and 2.17b show the relationship between the number of cycles and the stress range, S_r, for the 31.8 mm (1¼ in.) and 22.2 mm (7/8 in.) studs. A regression analysis of the test results yielded the following results:

For the 31.8 mm (1¼ in.):

$S_r = 355.0 - 40.1 \log (N)$ (SI Units) Equation 2.3a

$S_r = 51.49 - 5.81 \log (N)$ (English Units) Equation 2.3b

Where: S_r = stress range, MPa (ksi)

N = number of cycles

Coefficient of correlation = 0.919

Standard error = 1.948

F-observed = 65.64

F-critical = 4.67 with 95 percent confidence or 9.07 with 99 percent confidence

For the 22.2 mm (7/8 in.) studs:

$S_r = 352.7 - 40.8 \log (N)$ (SI Units) Equation 2.4a

$S_r = 51.15 - 5.92 \log (N)$ (English Units) Equation 2.4b

Coefficient of correlation = 0.902

Standard error = 1.946

F-observed = 39.083

F-critical = 5.12 with 95 percent confidence or 10.56 with 99 percent confidence

The "goodness of the fit" can be judged from **Figures 2.17a and 2.17b** where the test data are compared with Equations 2.3 and 2.4. The limits of dispersion were taken as twice the standard error of estimate and are shown in dashed lines parallel to the regression line in **Figures 2.17a and 2.17b.** Analyzing the regression results shows that both equations have high correlation with the test results, over 90 percent, and all of the test results are between the dispersion limits. Also, comparing the F-observed value with the F-critical value shows very high confidence level, 99 percent.

2.4.7 Proposed Fatigue Equation

The fatigue resistance force of shear connectors, Z_r, is given as follows:

$$Z_r = S_r\,(A) = \alpha\, d^2 \qquad \text{Equation 2.5}$$

Where A = the shear connector cross sectional area

$$\alpha = S_r \left(\frac{\pi}{4}\right) \qquad \text{Equation 2.6}$$

Based on the fatigue test results obtained in this research project, Equations 2.3 and 2.4, the following equation of α-value is recommended to be used:

<u>For the 31.8 mm (1¼ in.) studs:</u>

α (MPa) = 278.8 – 31.4 log (N) (SI Units) Equation 2.7a

α (ksi) = 40.44 – 4.56 log (N) (English Units) Equation 2.7b

<u>and for the 22.2 mm (7/8 in.) studs:</u>

α (MPa) = 277.0 – 32.1 log (N) (SI Units) Equation 2.8a

α (ksi) = 40.17 – 4.65 log (N) (English Units) Equation 2.8b

2.4.8 Comparison with the AASHTO Specifications

Section 10.38.5.1.1 of the AASHTO Standard Specifications (1996) gives the α-values as follows:

Number of Cycles (N)	"α" (MPa)	"α" (ksi)
100,000	89.6	13.00
500,000	73.1	10.60
2,000,000	54.1	7.85
> 2,000,000	38.0	5.50

Equation 6.10.7.4.2-2 of the LRFD Specifications (1998) gives the α-values as follows:

α (MPa) = 238 – 29.4 log (N) LRFD, EQ. 6.10.7.4.2-2, SI Units

α (ksi) = 34.5 – 4.28 log (N) LRFD, EQ. 6.10.7.4.2-2, English Units

Figure 2.18 gives a comparison of the α-values between the AASHTO Standard Specifications (1996), AASHTO LRFD Specifications (1998), and the current testing program (Equations 2.7 and 2.8). It is clear that the proposed equations yields 33 and 30 percent increase in the fatigue resistance for the 31.8 mm (1¼ in.) and 22.2 mm (7/8 in.) studs, respectively.

2.4.9 Lower Limit of the Proposed Equations

Since NDOR has adopted the AASHTO LRFD Specifications as its design specifications, the lower limit of the proposed equations will be discussed and developed in lieu of the LRFD Specifications.

In order to protect the shear connector against failure, two modes of failure should be addressed. The first mode is shear failure in the stud and the second mode is fatigue failure in the top flange of the steel girder. In the stud shear failure, crack starts at the toe of the weld and then propagates horizontally until the stud is sheared off. In the flange fatigue failure crack forms at the stud weld and propagates into the girder flange.

In high-traffic-volume bridges, the maximum number of cycles can be as high as 100,000,000 cycles as predicted by Equation 6.6.1.2.5-2 of the LRFD Specifications (1998). Therefore, Equation 6.6.1.2.5-1 of the LRFD Specifications states that the nominal fatigue stress resistance $(\Delta F)_n \geq \frac{1}{2}(\Delta F)_{TH}$, where $(\Delta F)_{TH}$ is the constantamplitude fatigue stress threshold and is given by Table 6.6.1.2.5-3 of the specifications. Using this approach, the LRFD Specifications provides a theoretically infinite fatigue life for the stud. Equation 6.10.7.4.2-1 of the LRFD Specifications set the following limit for $(\Delta F)_{TH}$:

$$Z_r = \alpha\ d^2 \geq \frac{38}{2} d^2 \qquad \text{(SI Units)} \qquad \text{Equation 2.9a}$$

$$Z_r = \alpha\ d^2 \geq \frac{5.5}{2} d^2 \qquad \text{(English Units)} \qquad \text{Equation 2.9b}$$

The LRFD has set the limit for Z_r to protect the girder top flange against fatigue failure. This decision was taken based on the extensive testing conducted by Fisher et al. (1990), where they found that girder fatigue failure controlled the stud capacity at high number of cycles.

Although the fatigue test results of this research project showed that only two specimens out of fourteen specimens failed due to fatigue cracking of the base plate, the research team did not feel comfortable in raising the lower limit that is given by the LRFD Specifications. This is because the relatively small number of cycles that were reached during the fatigue test (about 7,000,000 cycles) compared to the number of cycles that may be obtained by Equation 6.6.1.2.5-2 of the LRFD. Therefore, the researchers recommend using the lower limit currently given by the LRFD Specifications.

2.4.10 Recommendations

Figure 2.18 gives a comparison of the α-value, for the 31.8 mm (1¼ in.) studs, between the AASHTO Standard Specifications (1996), AASHTO LRFD Specifications (1998), and the current testing program (Equations 5 and 6).

From the comparison, the following conclusions can be drawn:

(1) It is conservative to use the α-values given by the AASHTO Standard and LRFD Specifications to calculate the allowable range of horizontal shear force for the 31.8 mm (1¼ in.) and the 22.2 mm (7/8 in.) studs.

(2) Designers are encouraged to use Equations 5 and 6 developed in this project. Using these equations will reduce the amount of required studs by about 30 percent, compared to the AASHTO Specifications, and will then reduce the initial cost of a bridge as well as the cost of deck removal.

(3) Because of the significant increase in the stud capacity when Equation 5 is used, it is expected that one row of 31.8 mm (1¼ in.) studs over the girder-web location, spaced at 152.4 mm (6 in.) or more, will be adequate to maintain full composite action for the great majority of bridges. This will increase safety of the construction workers and will ease deck replacement in the future.

(4) All specimens in this program were tested using a minimum stress of 34.48 MPa (5 ksi), which is higher than the 13.79 to 20.69 MPa (2 to 3 ksi) used in developing the AASHTO Standard and LRFD

Specifications equations. Thus, Equations 5 and 6 should predict more conservative fatigue results.

(5) Use the lower limit of the fatigue stud capacity that is given by the LRFD Specifications.

2.5 DEMONSTRATION BRIDGE

Nebraska Department of Roads (NDOR) assigned a three-span continuous bridge to implement the 31.8 mm (1¼ in.) studs. The bridge is on Highway 71 at Gering South, Scotts Bluff County, Nebraska.

2.5.1 Bridge Description

The bridge consisted of three continuous spans of 13.7, 18.3, and 13.7 m (45, 60, and 45 ft). Cross-section of the bridge consisted of five W30x99 rolled steel girders spaced at 2.7 m (8 ft –9 in.) supporting a 190 mm (7.5 in.) thick composite cast-in-place slab. Total width of the bridge is 12.6 m (41 ft –4 in.), as shown in **Figure 2.19**.

Preliminary design of the composite action included three 22.2 mm (7/8 in.) studs per row at spacing ranging from 254.0 to 406.4 mm (10 to 16 in.), as shown in **Figure 2.20**. NDOR agreed to implement the 31.8 mm (1¼ in.) studs on the south exterior span, while using the 22.2 mm (7/8 in.) studs on the center and north exterior span. This arrangement was chosen to give NDOR the opportunity to compare the structural performance between the 22.2 mm (7/8 in.) and 31.8 mm (1¼ in.) studs.

Using the 31.8 mm (1¼ in.) studs resulted in using one stud per row welded directly over the girder web with spacing ranging from 177.8 to 254.0 mm (7 to 10 in.).

Since the fatigue investigation of the 31.8 mm (1¼ in.) studs was not completed by the time the bridge was constructed and this was the first project where the 31.8 mm (¼ in.) studs were implemented, NDOR Bridge Engineers were very cautious and decided to:

1. Use the α-values of fatigue capacity as given in the AASHTO Standard Specifications, which resulted in one stud per row welded directly over the girder web location with spacing ranging from 177.8 to 254.0 mm (7 to 10 in.), as shown in **Figure 2.20b.**
2. Use only headed studs.
3. Add a washer to the stud assembly. The washer was 4.8-mm (3/16-in.) thick, had an outside diameter of 76.2 mm (3 in.), and was tack welded to the nut, as shown in **Figures 2.21 and 2.22**. The washer was added to the stud assembly to ensure confinement of the concrete around the stud. However, upon completion of the fatigue test program, it was evident that providing continuous top and bottom reinforcement in the deck slab can provide adequate confinement to the concrete around the stud. Thus, the researchers have advised NDOR not to add the washer to the stud in future projects.

After letting the project, NDOR called for a change order from 22.2 mm (7/8 in.) studs on the south exterior span to 31.8 mm (1¼ in.) studs. The 31.8 mm (1¼ in.) studs were produced by Try Sales Associates, Omaha, Nebraska (see Appendix A). Studs were welded by the steel fabricator, Capital Contractors, in their shop in Lincoln, Nebraska. An electric source of 2,500 Amperage was used in welding the studs. The stud welder made some trials before welding the 31.8 mm (1¼ in.) on the steel girders, where he became very confident that he could weld the studs without using the tripod support.

Welding of the studs on the steel girders went at a rate of 40 seconds per stud without any problems. The quality control test, as described in Section 2.3 of this report, was conducted at three locations on each girder. Even though a factor of safety of one was used in the quality control test, no welding failure was observed during the quality control test. Although successful attempts were made by the welding technician to test the weld quality of the 31.8 mm (1¼ in.) by bending it to 45 degrees as shown in **Figure 2.23**, it took a lot of time and manpower that proved to everyone that this is not a practical way to test these large studs. **Figure 2.24** shows the 31.8 mm (1¼ in.) studs after welding where perfect round was achieved and **Figure 2.25** shows the stud after adding the nut and the washer.

2.5.2 Bridge Construction

The bridge construction was completed in the fall of 1999. **Figure 2.26** shows a top view of the bridge deck on the south span before pouring the cast-in-place concrete. The bridge contractor commented that using one line of studs provided the construction crew with higher level of safety during deck construction and that welding the studs in the steel fabricator's shop increased the construction speed and produced high welding quality for the studs. **Figures 2.27 and 2.28** show the top and side views of the bridge after construction was completed. The bridge will be opened to traffic in 2000.

2.5.3 Deflection Measurements

The researchers, jointly with NDOR engineers, took deflection measurements of the bridge using a three-axle dump truck. The truck was positioned at three locations (load cases) in the longitudinal direction and the deflection was measured at each location. For all locations, the truck was positioned transversely such that one line of the wheels was directly over the center girder and the other line was between the center girder and the first interior girder as shown in **Figure 2.29-a**. The three locations were as follows (see **Figure 2.29-b**):

Location (load case) #1: the truck was positioned on the South exterior span (Southbound) of the bridge such that the center of gravity of the three axles was at a distance of 5.5 m (18 ft) from the south abutment. Deflection measurement of the center girder at this point was 3 mm (0.118 in.).

Location (Load case) #2: the truck was positioned on the center span such that the center of gravity of the three axles was at a distance of 9.1 m (30 ft) from the south pier. Deflection measurement of the center girder at this point was 2.5 mm (0.098 in.).

Location (Load case) #3: the truck was positioned on the North exterior span (Northbound) of the bridge such that the center of gravity of the three axles was at a distance of 8.2 m (27 ft) from the North abutment. Deflection measurement of the center girder at this point was 3 mm (0.118 in.).

Visual inspection of the bridge deck showed no cracks or distress on the south exterior span where the 31.8 mm (1¼ in.) studs were used.

2.5.3.1 Comparing Field Measurements with Theory

The lever rule was used to calculate the distribution factor for the center girder. From **Figure 2.29-a**, reaction at the center girder equals $0.5P + 0.5P(\frac{2.75}{8.75}) = 0.657P$.

Thus, the distribution factor for moment equals 0.657

Composite properties of the center girder are as follows:

Cross section area = 79742 mm^2 (123.6 in^2)

Moment of inertia = 5.04035×10^9 mm^4 (12,109.5 in^4)

Using continuous beam analysis software, deflection due to the three loading cases using a distribution factor of 0.657 are as follows:

Location (Load case)	Field deflection measurement mm (in)	Theoretical deflection mm (in)
1	3 (0.118)	4.7 (0.187)
2	2.5 (0.098)	8.0 (0.316)
3	3 (0.118)	4.3 (0.171)

Field deflection measurements of the south exterior span bridge (where the new studs were used) were almost the same as that of the north exterior span (where the conventional studs are used). This showed that the new studs have no harmful effect on the deflection of the bridge, i.e. no reduction in the composite action properties of the bridge. Also, field deflection measurements were smaller than those calculated by theory, which means that the actual structure is stiffer than that what is considered in the design calculations.

2.6 COST ANALYSIS

Cost analysis given in this section includes incremental cost of using the 31.8 mm (1¼ in.) studs for the demonstration bridge and the expected incremental cost for future projects.

2.6.1 Incremental Cost of the Demonstration Bridge

Original plans of the bridge included 135- 22.2 mm (7/8 in.) studs per girder on the South exterior span at a price of $0.71 per stud. This price does not include the cost of welding the studs to the steel girder.

Thus, material cost of the 22.2 mm (7/8 in.) studs for this span

= 135 x 0.71 = $95.85 per girder = 95.85 x 5 girders = $479.25 per span

Replacing the 22.2 mm (7/8 in.) studs with the 31.8 mm (1¼ in.) studs resulted in using 67- 31.8 mm (1¼ in.) studs per girder. Cost of the new stud was as follows:

31.8 mm (1¼ in.) threaded stud	= $3.45/pc
Heavy hexagonal nut	= $1.05/pc
Flat washer	= $0.42/pc
Welding the washer to the nut	= $0.25/pc
Total	= $5.17/pc

Material cost of the 31.8 mm (1¼ in.) studs = 5.17 x 67 = $346.39 per girder

= 346.39 x 5 girders = $1,731.95 per span

Thus, incremental cost = 1,731.95 – 479.25 = $1,252.70

= $1,252.70 / (41.34 ft x 45 ft)

= $0.67 per ft^2 ($7.25 per m^2)

Notes:

Using the 31.8 mm (1¼ in.) studs in the demonstration project resulted in incremental cost for the following reasons:

1. NDOR bridge engineers wanted the 31.8 mm (1¼ in.) studs to be produced by the same manufacture that produced studs for the NCHRP 12-41 Project. Thus, no

competitive price for the 31.8 mm (1¼ in.) studs was provided for the demonstration project.

2. Adding the washer raised the cost by $0.67 per stud. Fatigue investigation showed that the washers are not needed. Thus, in future projects, this item will be eliminated.
3. Fatigue capacity of the studs was determined using the α-values given in the AASHTO Standard Specifications, which are shown by the fatigue investigation to be conservative. In future projects, it is recommended to use the α-equations that were developed in this research project. This will result in reducing the number of the 31.8 mm (1¼ in.) studs by 30 percent.
4. Saving in labor due to the reduction in the number of welded studs is not considered.
5. Saving in time and labor associated with deck removal in the future is not considered.

2.6.2 Estimated Incremental Cost for Future Projects

UNL researchers believe that the studs ordered for the demonstration bridge were over priced due to lack of competition. Therefore, UNL researchers investigated other sources for producing the 31.8 mm (1¼ in.) studs. One source was found, Continental Studwelding Ltd. (see Appendix A). In order to reduce the production price of the 31.8 mm (1¼ in.) studs, the supplier suggested that a headed stud, as shown in **Figure 2.30**, be produced by using a hot forge machine. By doing that, all the labor that is associated with threading the stud and using a heavy and expensive nut is eliminated. The following are excerpts from the supplier's response to the UNL researcher questions:

Q1. What type of steel will be used in producing the 31.8 mm (1¼ in.) studs?

A1. The steel used in the production of 31.8 mm (1¼ in.) studs will conform to ASTM A108 grades SAE 1010 through SAE 1020. Either SAE 1015 or SAE 1018 will be the most commonly used material.

Q2. Can the 31.8 mm (1¼ in.) be produced as headed stud?

A2. Yes, the 31.8 mm (1¼ in.) diameter studs would be produced as headed studs. The head diameter can be as large as two times the stud diameters, 63.5 mm (2½ in.).

Q3. How will the 31.8 mm (1¼ in.) studs be produced?

A3. These studs would be formed using a hot forge process. First, blanks (31.8 mm (1¼ in.) diameter bars without heads) will be created. These bars would pass through an oven where one end of each blank will be heated through the application of a flame produced by a mixture of natural gas and pressurized air. These heated blanks would be put into a forging press machine, which would hit the hot end of the blank to form the head. The finished studs would be air cooled prior to packaging.

Q4. What is the expected cost of the 31.8 mm (1¼ in.) studs?

A4. Estimated price for producing the 31.8 mm (1¼ in.) stud is $2.09 per stud. This price is based on a quantity of 2000 studs without including shipping cost.

Based on this investigation, expected incremental cost for future project can be illustrated by considering the exterior span of the demonstration bridge as follows:

- Using the α-values, given by the AASHTO Specifications, and 22.2 mm (7/8 in.) studs at \$0.71 per stud, resulted in 135 studs at a total cost of = 135 x \$0.71 = \$95.85 per girder = \$95.85 x 5 girders = \$479.25 per span
- Using 31.8 mm (1¼ in.) studs at \$2.09 per stud and the α-values as recommended from the fatigue investigation of this research will result in

  = 135 studs x 0.5 (31.8 mm (1¼ in.) stud replaces 2- 22.2 mm (7/8 in.) studs) / 1.3 (30% increase in fatigue capacity)

  = 52 (31.8 mm (1¼ in.) studs) x \$2.09 = \$108.68 per girder

  = \$108.68 x 5 girders = \$543.40 per span
- Incremental cost = 543.40 – 479.25 = \$64.15 per span

  = \$64.15 / (41.34 ft x 45 ft)

  = \$0.034 per ft^2 (\$0.37 per m^2)

With repeated use of the 31.8 mm (1¼ in.) studs, this minimal premium will actually turn into savings. The level of savings is dependent on the volume and consistency of usage.

2.7 CONCLUSIONS

Based on the testing program conducted in this project and in NCHRP Project 12-41, the following conclusions can be drawn:

2.7.1 Advantages

- It is expected that by using the 31.8 mm (1¼ in.) studs, with one row of studs over the girder-web location, spaced at 150 mm (6 in.) or more, will be adequate to maintain full composite action for the great majority of bridges. This will facilitate deck removal, improve the bridge economy and construction speed, and improve safety for the construction workers.

2.7.2 Stud Material, Dimensions, and Welding

1. 31.8 mm (1¼ in.) studs can be produced as headed or headless studs.
2. Cold drawn Society of Automotive Engineers (SAE) 1015 or 1018 steel can be used to produce the 31.8 mm (1¼ in.) shear studs.
3. Arc stud welding can be used for welding of the 31.8 mm (1¼ in.) studs.
4. If needed, a tri-leg support may be used to help in adjusting the stud verticality during welding.
5. Steep stud chamfer, large amount of flux, and a minimum power supply of 2,500 Amperage are needed for good welding quality.
6. Welding quality can be assessed by using the proposed quality control test, i.e. shearing two adjacent studs using a small hydraulic jack.

2.7.3 Stud Ultimate Capacity

1. AASHTO LRFD Equation 6.10.7.4.4c-1 can be used to predict the 31.8 mm (1¼ in.) stud ultimate capacity. Using this equation will result in replacing two 22.2 mm (7/8 in.) studs with one 31.8 mm (1¼ in.) stud.

   $Q_n = 0.5\, A_{SC} \sqrt{f'_c E_c} \leq A_{SC} F_u$ LRFD EQ. 6.10.7.4.4c-1, SI Units

   $Q_n = 0.5\, A_{SC} \sqrt{f'_c E_c} \leq A_{SC} F_u$ LRFD EQ. 6.10.7.4.4c-1, English Units

2. The center-to-center pitch of the 31.8 mm (1¼ in.) studs shall be not less than 152.4 mm (6.0 in.).
3. It is recommended to use continuous top and bottom transverse reinforcement over the girder lines to provide adequate confinement of the concrete around the stud.
4. Cyclic loading has no detrimental effect on the ultimate capacity of the 31.8 mm (1¼ in.) stud.
5. Residual stresses due to replacement of the 22.2 mm (7/8 in.) studs with 31.8 mm (1¼ in.) studs appeared to have no detrimental effect on the capacity of the 31.8 mm (1¼ in.) stud.
6. Replacing 50 percent of the 31.8 mm (1¼ in.) headed studs with headless studs resulted in a reduction of the stud capacity of 17 percent. It is difficult to estimate the reduction when headed studs are fully replaced with headless studs.
7. Although use of 31.8 mm (1¼ in.) headless studs may appear to be disadvantageous due to the anticipated reduction in stud capacity, ease of long-term deck removal and replacing may more than compensate for the increased number of headless studs used. Further study is needed in this area.

2.7.4 Stud Fatigue Capacity

1. AASHTO LRFD Equation 6.10.7.4.2-1 can be conservatively used to predict the 31.8 mm (1¼ in.) and the 22.2 mm (7/8 in.) stud fatigue capacity.

   SI Units:

   $$Z_r = \alpha\ d^2 \geq \frac{38}{2}\ d^2 \qquad \text{LRFD Equation 6.10.7.4.2-1}$$

$\alpha = 238 - 29.4 \text{ Log(N)}$ LRFD Equation 6.10.7.4.2-2

English Units:

$Z_r = \alpha \ d^2 \geq \frac{5.5}{2} d^2$ LRFD Equation 6.10.7.4.2-1

$\alpha = 34.5 - 4.28 \text{ Log(N)}$ LRFD Equation 6.10.7.4.2-2

2. Designers are encouraged to use the following equations that are developed in this project for the 31.8 mm (1¼ in.) and 22.2 mm (7/8 in.) studs:

SI Units:

$$Z_r = \alpha \ d^2 \geq \frac{38}{2} d^2$$

$\alpha = 278.8 - 31.4 \log (N)$ (for the 31.8 mm (1¼ in.) studs)

$\alpha = 277.0 - 32.1 \log (N)$ (for the 22.2 mm (7/8 in.) studs)

English Units:

$$Z_r = \alpha \ d^2 \geq \frac{5.5}{2} d^2$$

$\alpha = 40.44 - 4.56 \log (N)$ (for the 31.8 mm (1¼ in.) studs)

$\alpha = 40.17 - 4.65 \log (N)$ (for the 22.2 mm (7/8 in.) studs)

Using these equations will reduce the amount of required studs by about 30 percent, compared to the AASHTO LRFD Specifications, which will then reduce the initial cost of a bridge, as well as the cost of deck removal.

3. It is recommended that the 31.8 mm (1¼ in.) studs be welded on steel plates that have a minimum thickness of 19 mm (¾ in.).

Table 2.1 Mechanical Properties of SAE 1018 and 1008

	SAE 1008 22.2 mm (7/8 in.) stud	SAE 1018 31.8 mm (1¼ in.) stud
Minimum Tensile Strength, MPa (psi)	340 (49,000)	440 (64,000)
Minimum yield strength, MPa (psi)	290 (41,5000)	370 (54,000)
Minimum elongation %	20	15
Minimum reduction in area %	45	40
Brinell hardness	95	126

Table 2.2 Stud Fatigue Capacity (α-values) in MPa (ksi)

	100,000 cycles	500,000 cycles	2,000,000 cycles
AASHTO Standard	95.2 (13.8)	73.1 (10.6)	54.1 (7.85)
AASHTO LRFD	90.3 (13.1)	69.6 (10.1)	51.7 (7.5)
Kakish (1997)	116.5 (16.9)	107.1 (15.53)	97.6 (14.15)

Table 2.3 Stress Range used in Previous Testing

Source	Test type	Stress range MPa (ksi)
Thurlimann (1959)	Push-off specimens	89.6 to 131.0 (13 to 19)
Thurlimann (1958)	Beam test	96.5 to 137.9 (14 and 20)
Culver et al (1961)	Beam test	151.7 (22)
King et al (1965)	Beam test	117.2 to 165.5 (17 to 24)
Toprac (1965)	Beam test	72.4 to 122.7 (10.5 to 17.8)
Slutter and Fisher (1966)	Push-off specimens	55.2, 82.7, 110.3, 137.9, and 165.5 (8, 12, 16, 20, and 24)

Table 2.4 Testing Plans of the 31.8 mm (1¼ in.) and the 22.2 mm (7/8 in.) Studs

Specimen Designation		Stress range, S_r MPa (ksi)	Minimum Stress MPa (ksi)
31.8 (1¼ in.) Stud	22.2 (7/8 in.) Stud		
LS-5-25-A LS-5-25-B	SS-5-25	172 (25) 172 (25)	34 (5) 34 (5)
LS-5-23-A LS-5-23-B	SS-5-23	159 (23) 159 (23)	34 (5) 34 (5)
LS-5-21	SS-5-21	145 (21)	34 (5)
LS-5-20	SS-5-20	138 (20)	34 (5)
LS-5-19	SS-5-19	131 (19)	34 (5)
LS-5-18-A LS-5-18-B	SS-5-18	124 (18) 124 (18)	34 (5) 34 (5)
LS-5-17	SS-5-17	117 (17)	34 (5)
LS-5-16	SS-5-16	110 (16)	34 (5)
LS-5-15	SS-5-15	103 (15)	34 (5)
LS-5-14		97 (14)	34 (5)
LS-5-10-A LS-5-10-B	SS-5-10	69 (10) 69 (10)	34 (5) 34 (5)

Table 2.5 Test Results of the Push-off Specimens

31.8 mm (1¼ in.) Stud Specimen			22.2 mm (7/8 in.) Stud Specimen		
Specimen	No. of Cycles	Type of failure	Specimen	No. of Cycles	Type of failure
LS-5-25A LS-5-25B	49,920 50,000	Stud failure Stud failure	SS-5-25	27,836	Stud failure
LS-5-23A	74,000	Stud failure	SS-5-23	60,000	Stud failure
LS-5-21	94,336	Stud failure	SS-5-21	285,315	Stud failure
LS-5-20	553,800	Stud failure	SS-5-20	188,760	Stud failure
LS-5-19	566,821	Concrete crushing & stud failure	SS-5-19	157,162	Stud failure
LS5-18A LS5-18B	166,000 2,533,000	Concrete crushing Stud failure	SS-5-18	935,391	Stud failure
LS5-17	1,636,000	Base plate failure	SS-5-17	400,000	Concrete crushing
LS-5-16	1,372,472	Base plate failure	SS-5-16	2,452,055	Stud failure
LS-5-15	2,000,000	No failure	SS-5-15	600,000	Stud failure
LS-5-14	2,594,000	No failure	SS-5-14	2,000,000	No failure
LS-5-10A LS-5-10B	4,680,000 6,708,000	No failure No failure	SS-5-10	2,500,000	No failure

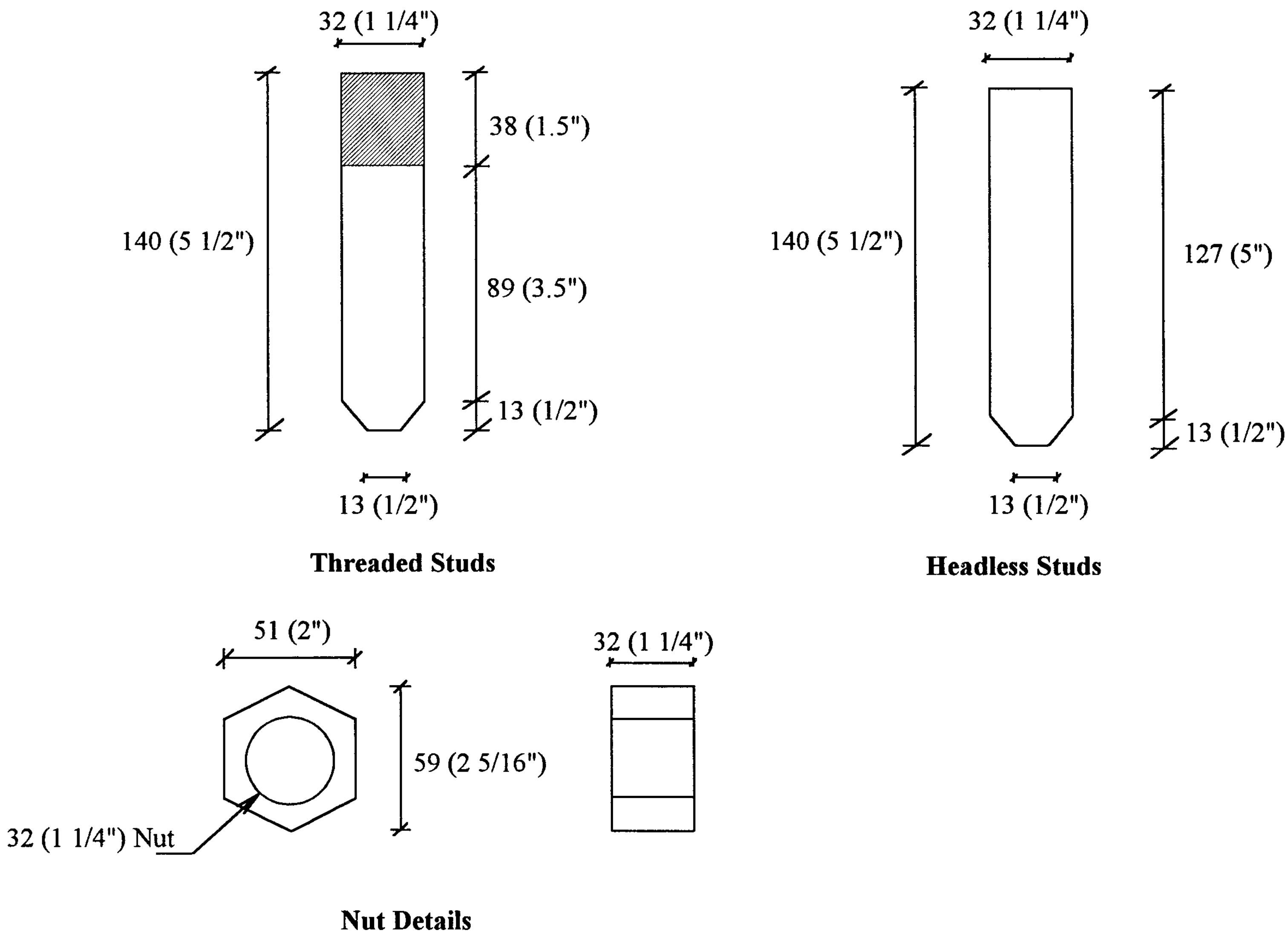


Figure 2.1 Details of the Proposed 31.8 mm (1¼ in.) Stud

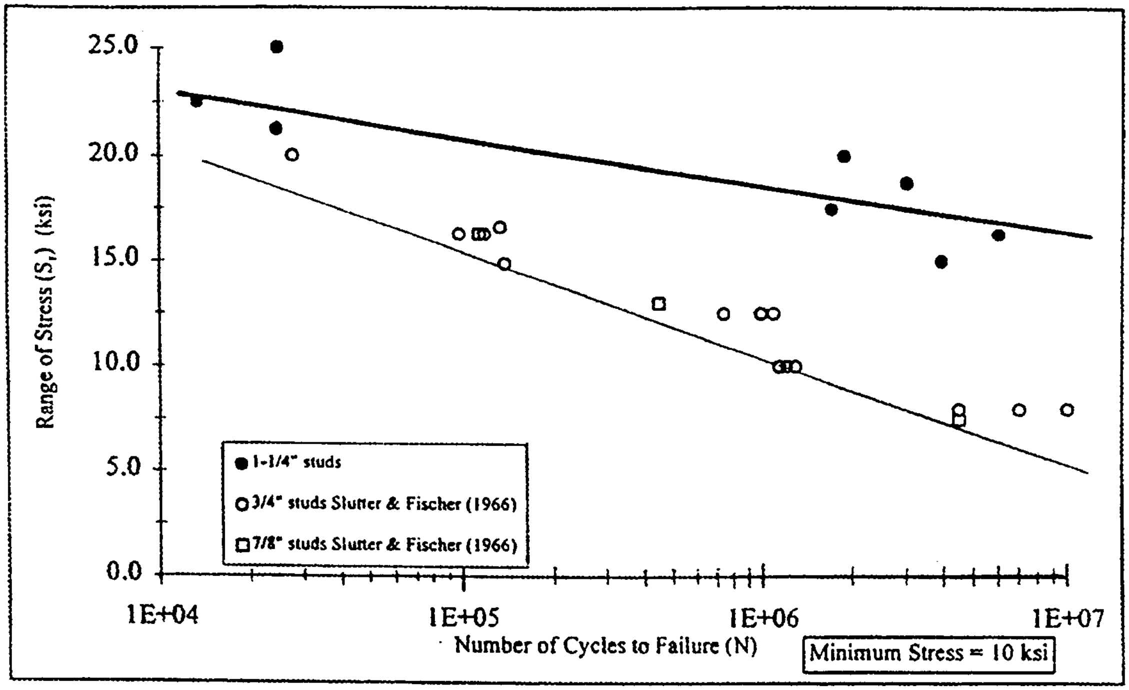
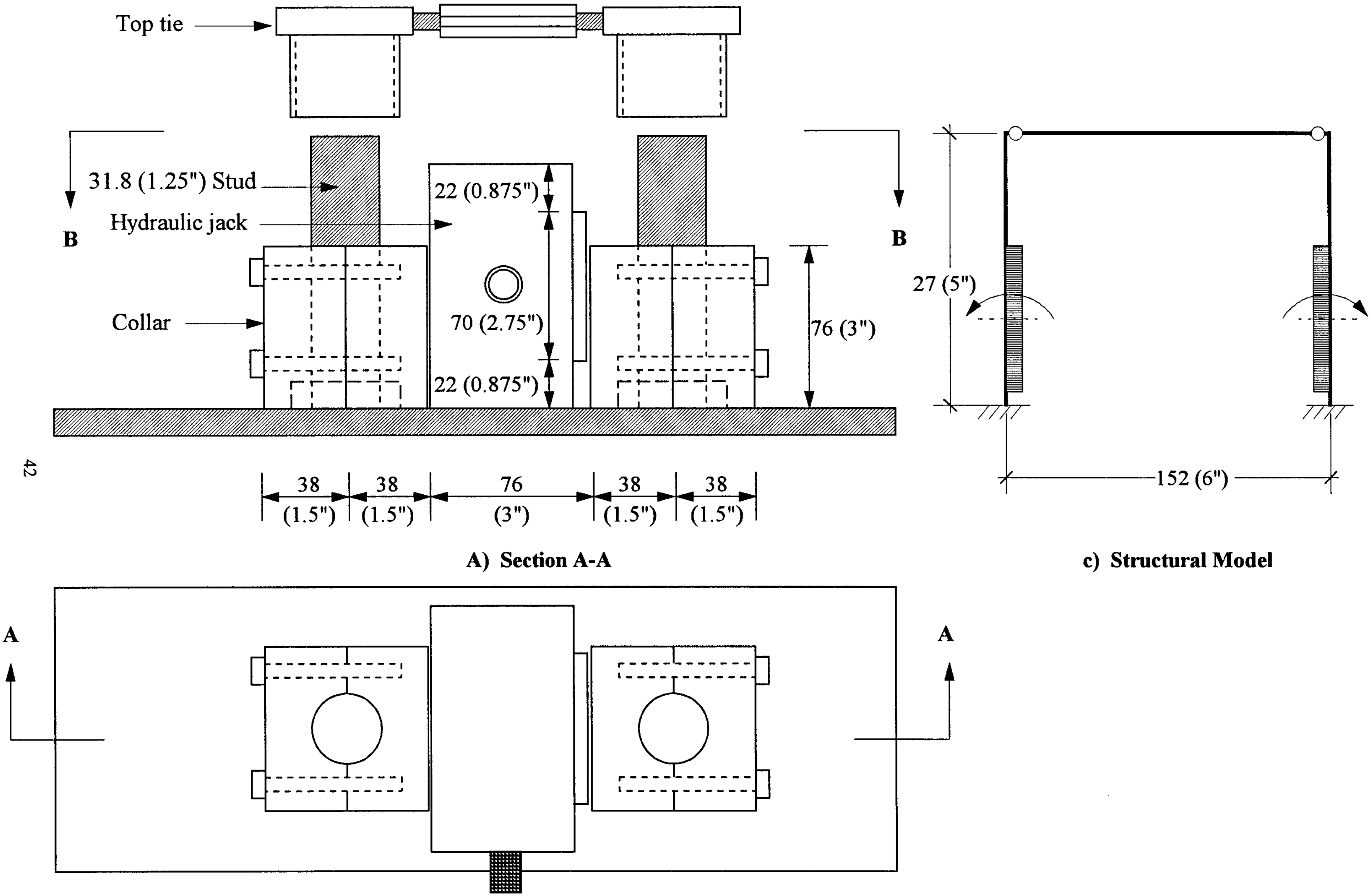


Figure 2.2 Stud Fatigue Results Given by Kakish (1997)



A) Section A-A

c) Structural Model

b) Section B-B

Figure 2.3 Schematic of the Quality Control Test Setup

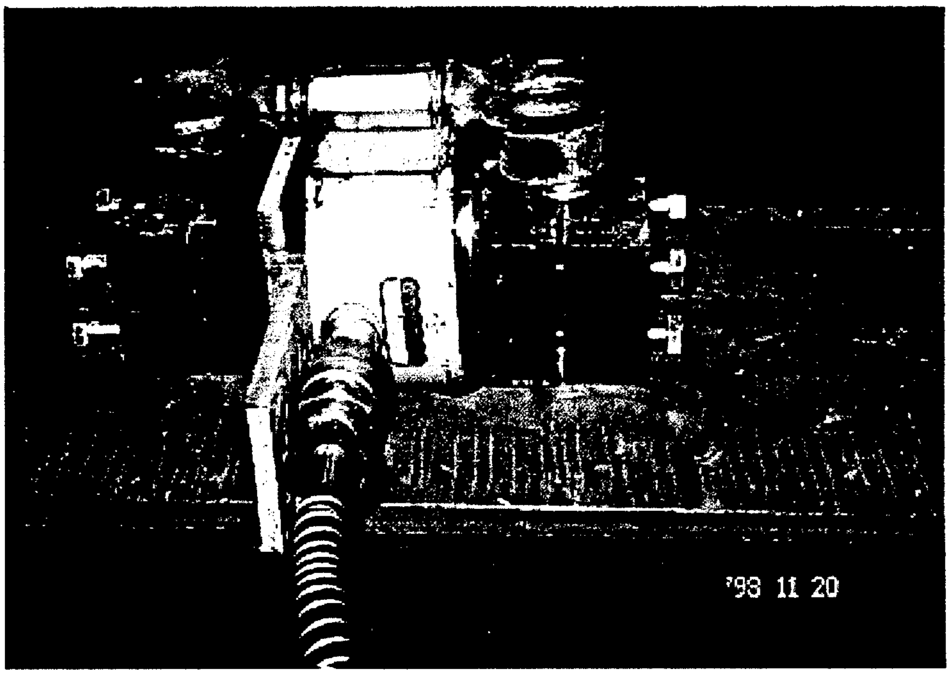


Figure 2.4 Quality Control Setup

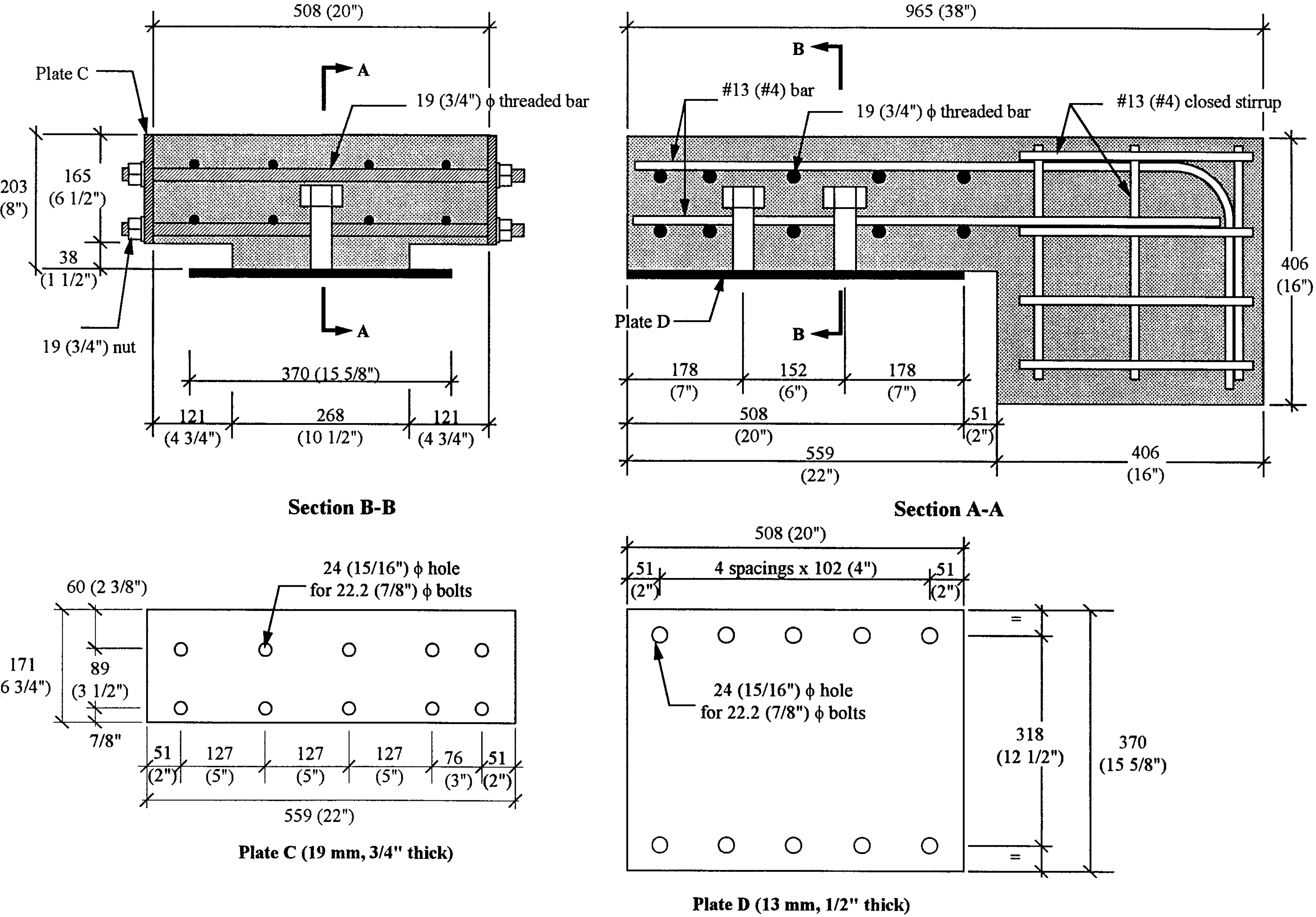
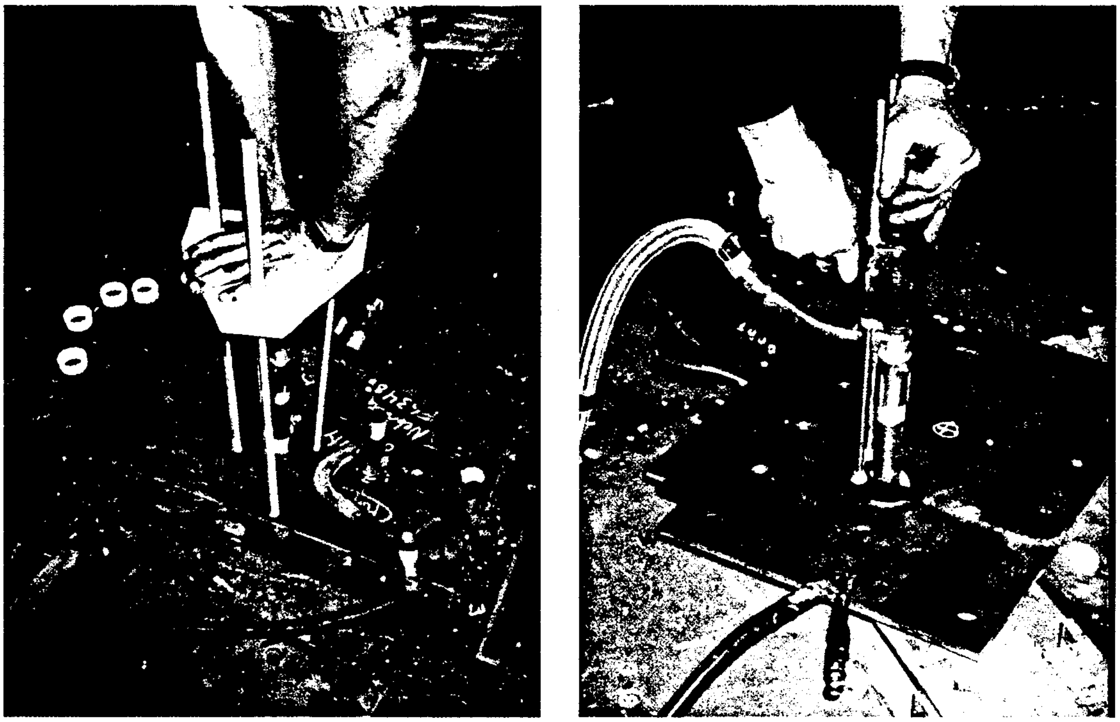


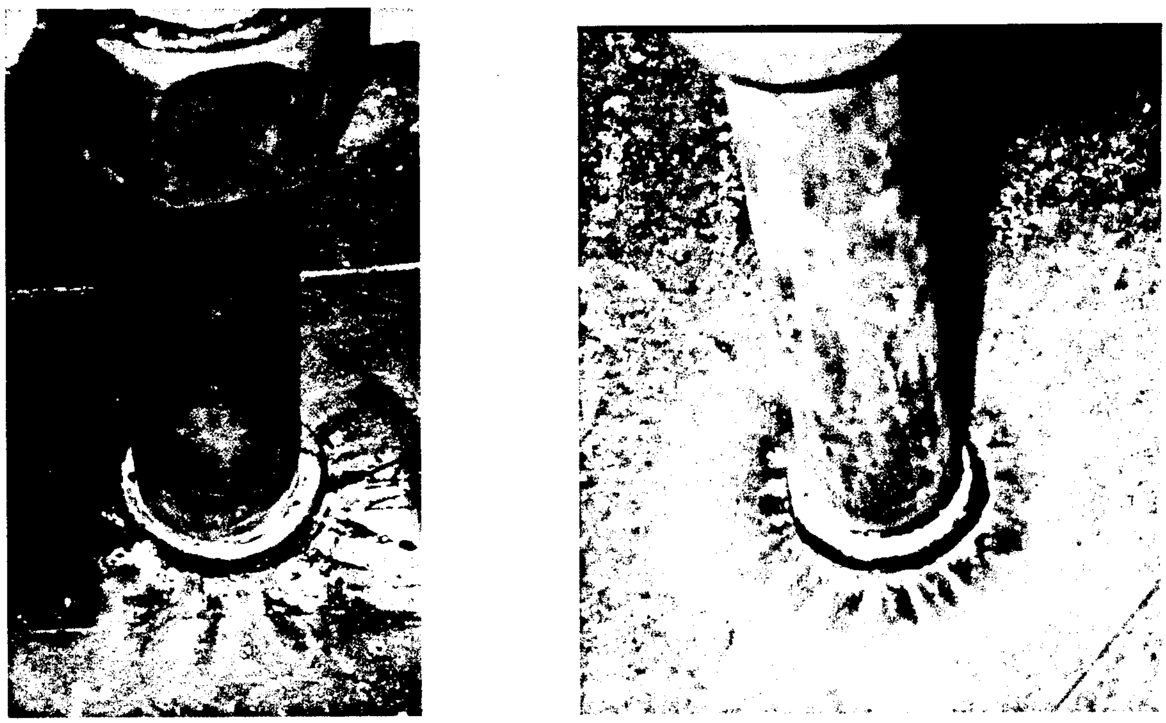
Figure 2.5 Details of the Fatigue Test Specimen



a) With the Tri-Leg Support

b) Without the Tri-Leg Support

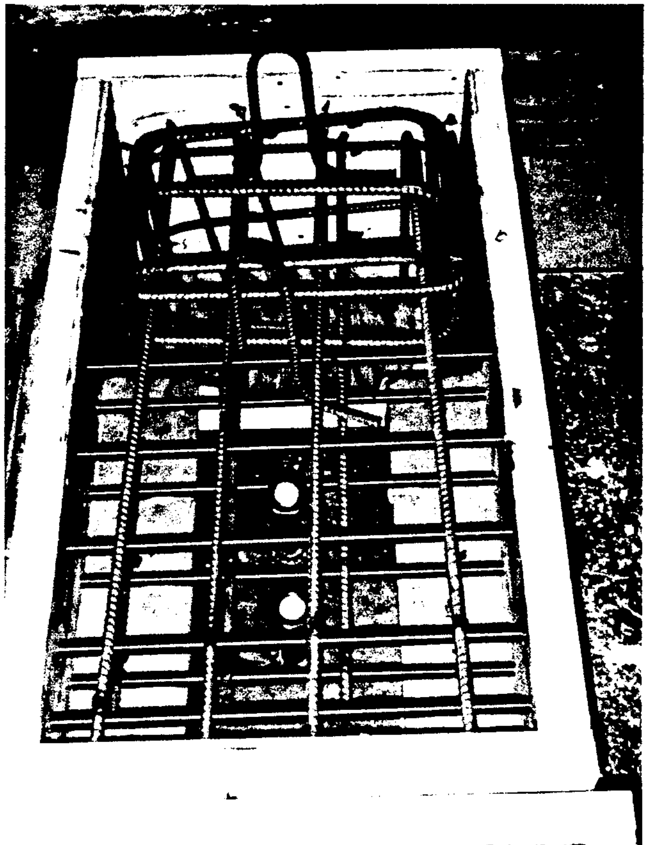
Figure 2.6 Welding of the 31.8 mm (1¼ in.) Stud



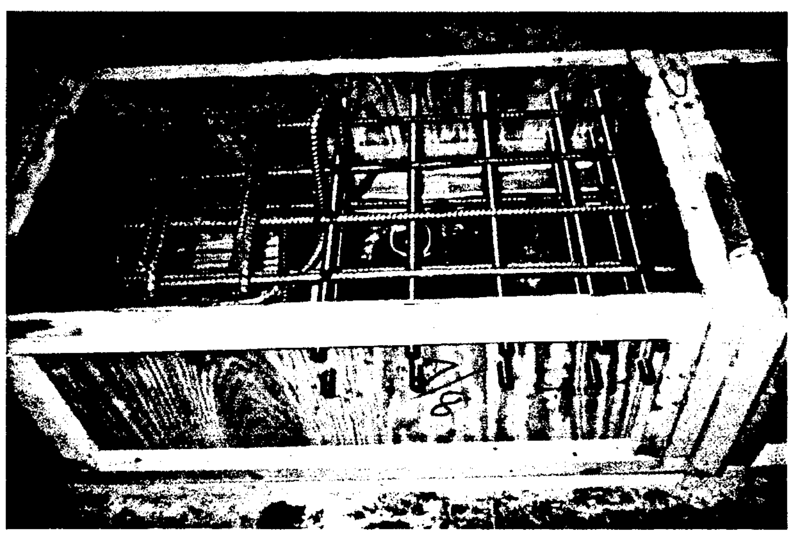
a) 31.8 mm (1¼ in.) Stud

b) 22.2 mm (7/8 in.) Stud

Figure 2.7 Stud Welding Quality



a) With 31.8 mm (1¼ in.) Studs



b) With 22.2 mm (7/8 in.) Studs

Figure 2.8 Forming of the Specimens

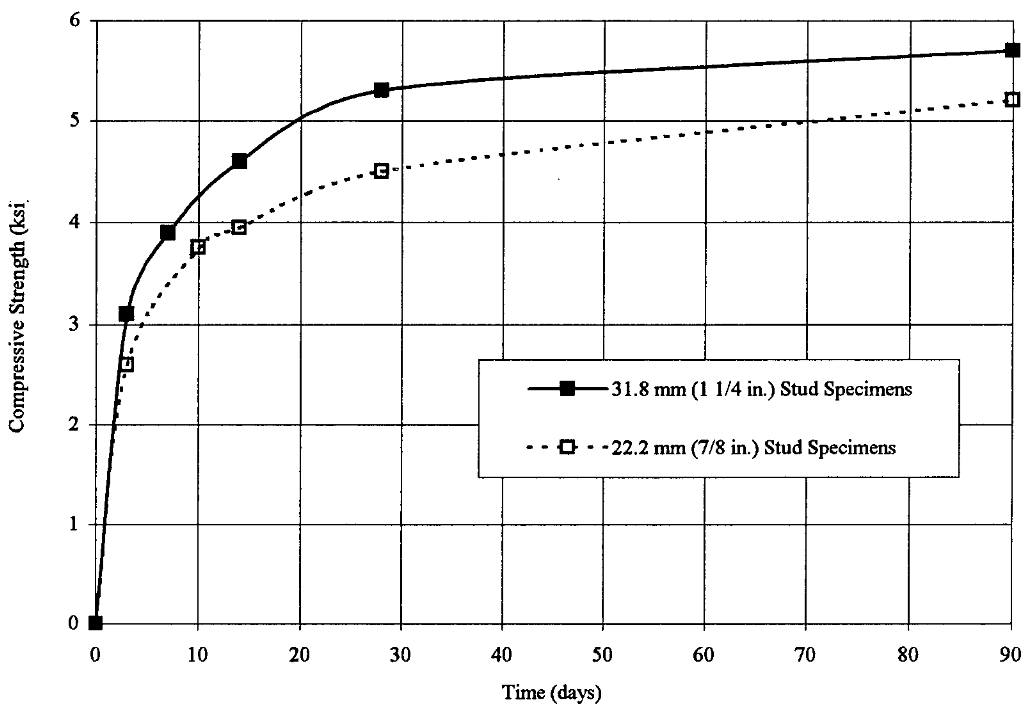


Figure 2.9 Compressive Strength vs. Time of the Concrete Mixes

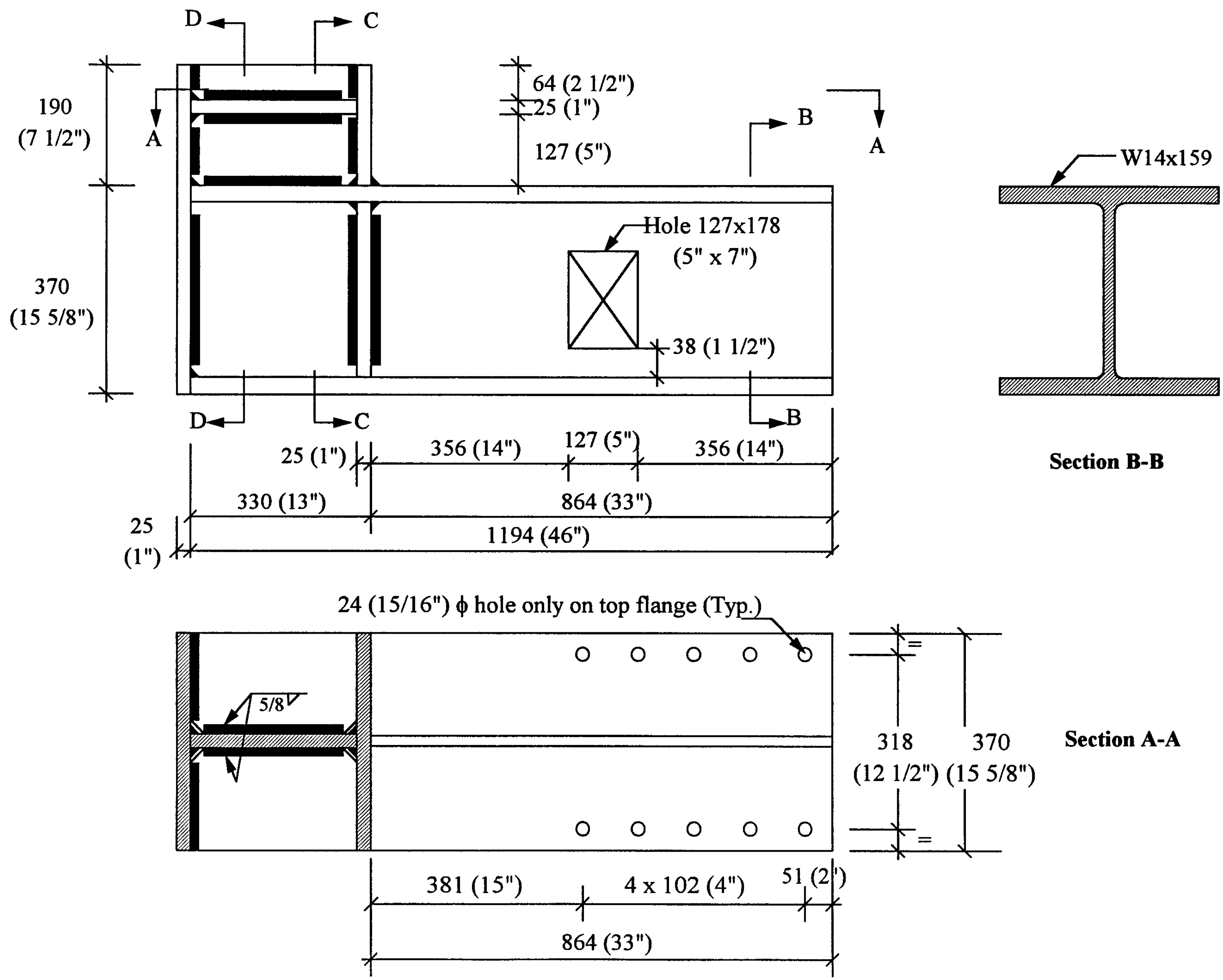


Figure 2.10 Details of the L-Beam

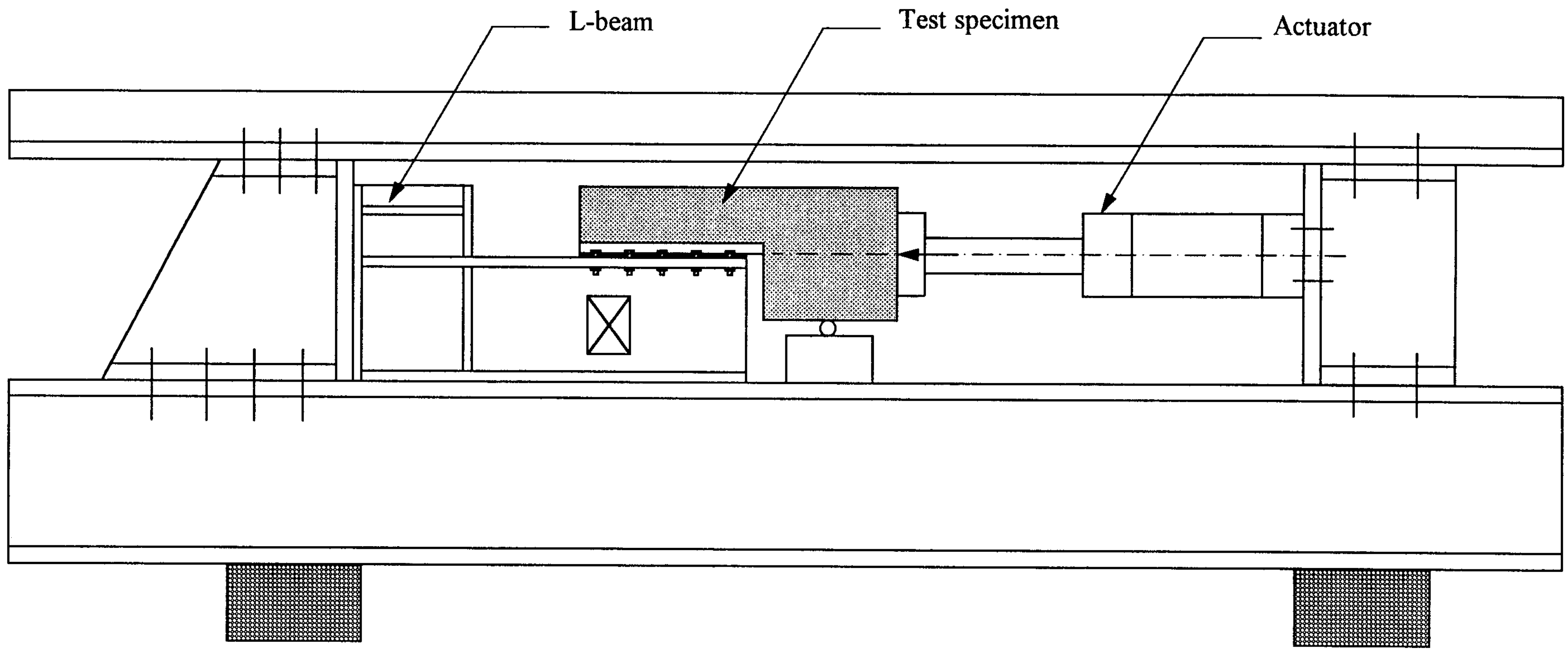


Figure 2.11 Test Setup

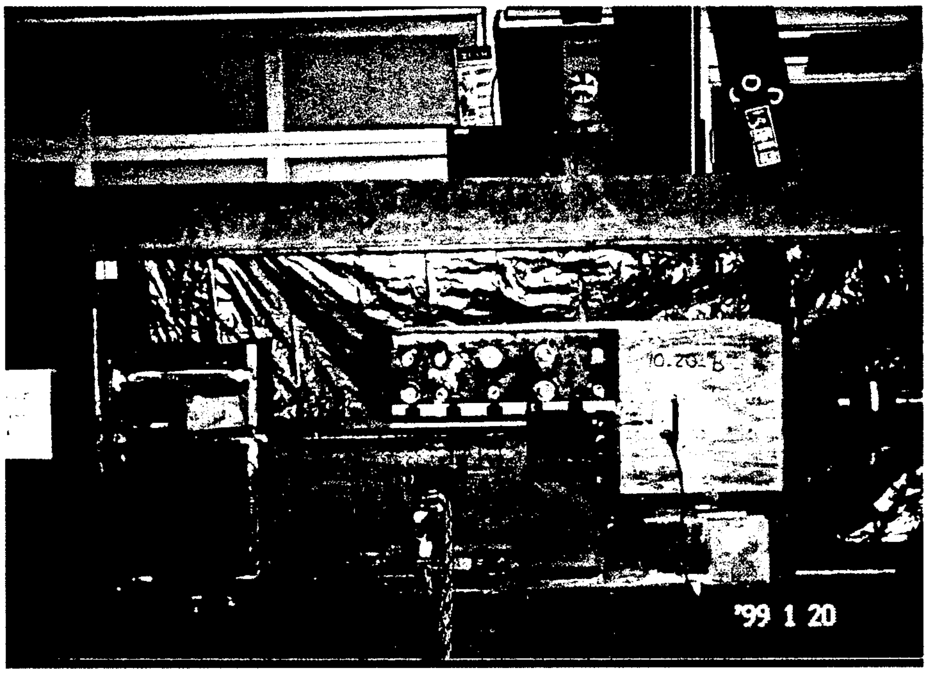
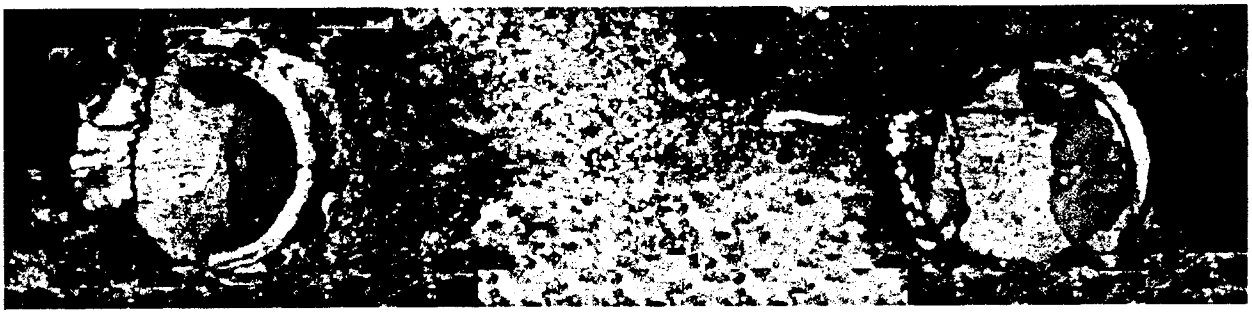


Figure 2.12 A Push-off Specimen during Testing



a) 31.8 mm (1¼ in.) Stud

b) 22.2 mm (7/8 in.) Stud

Figure 2.13 Fatigue Failure of the Studs

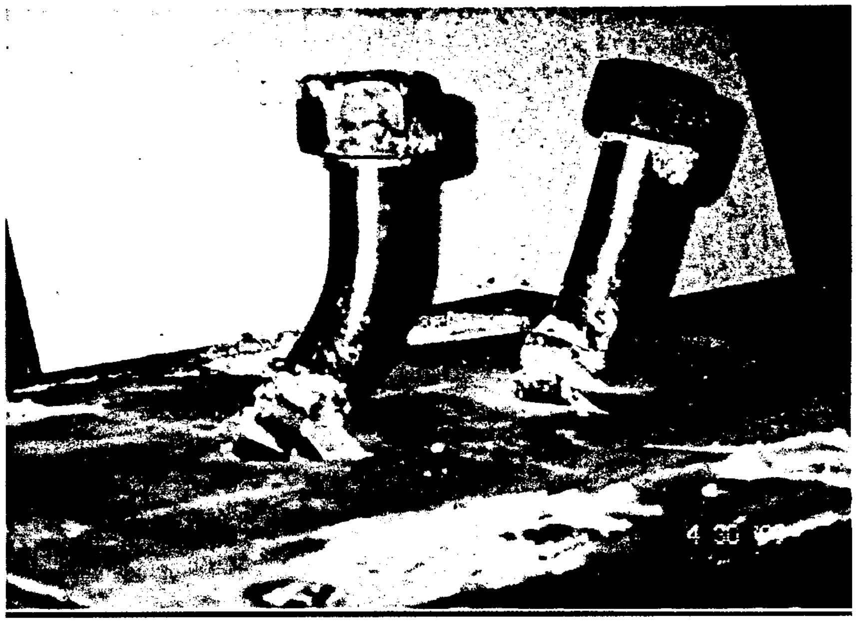
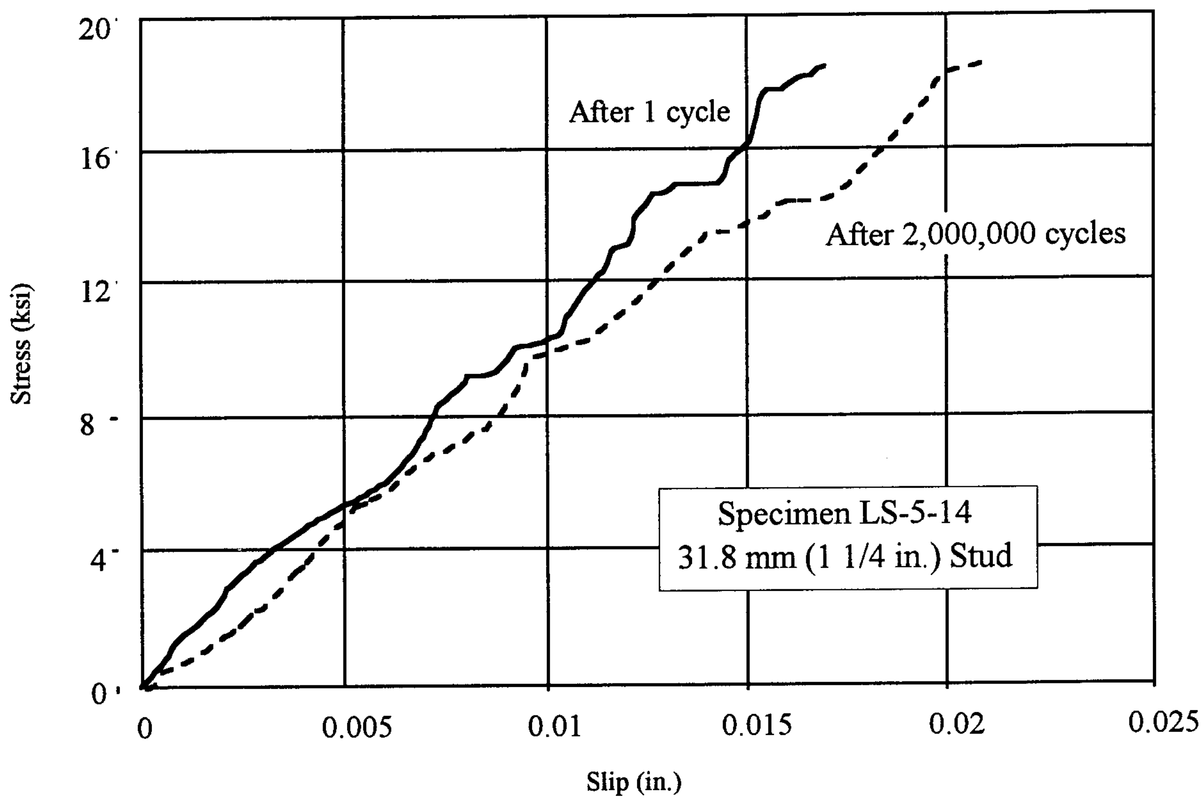


Figure 2.14 Base-Plate Failure

Figure 2.15 Concrete Failure



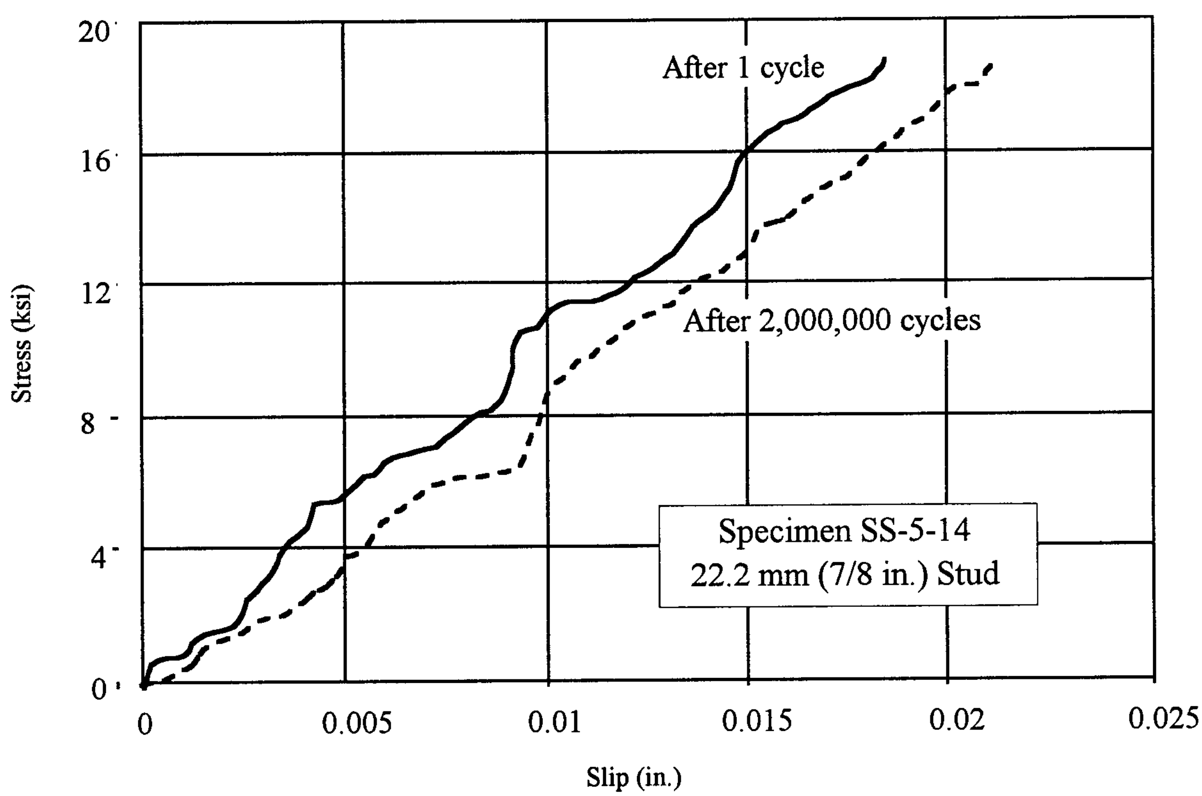
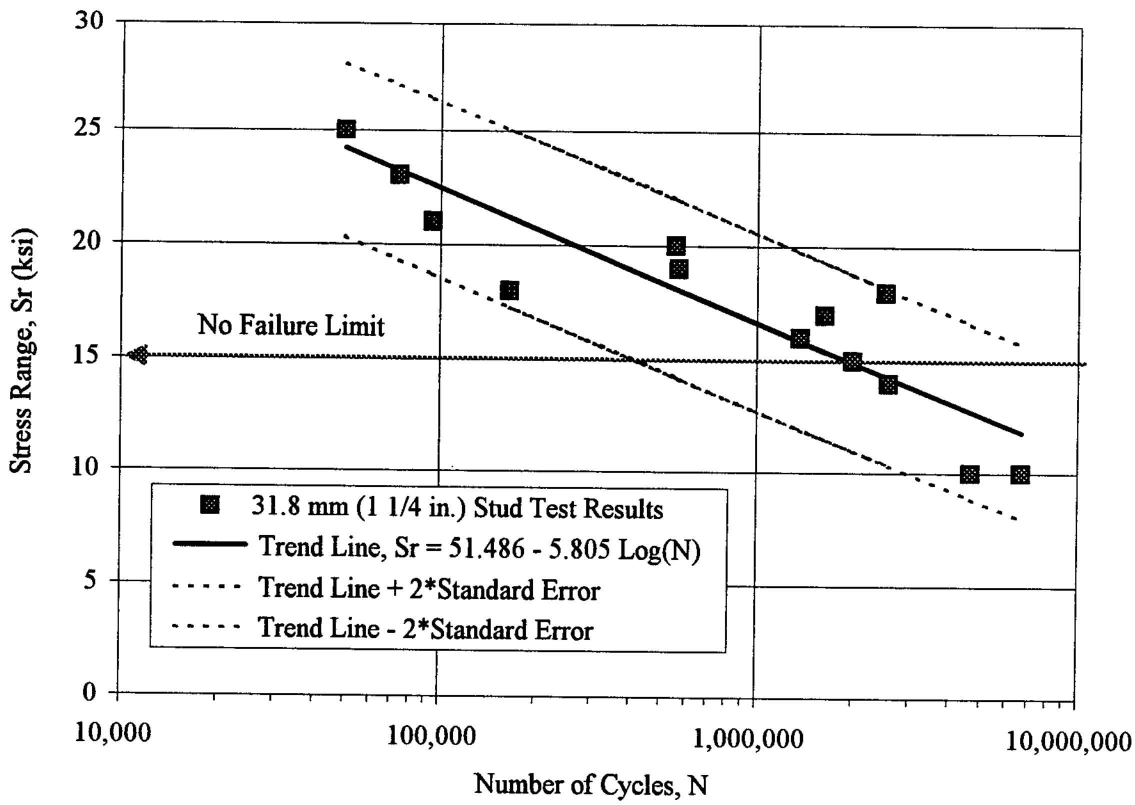


Figure 2.16 Slippage vs. Horizontal Stress



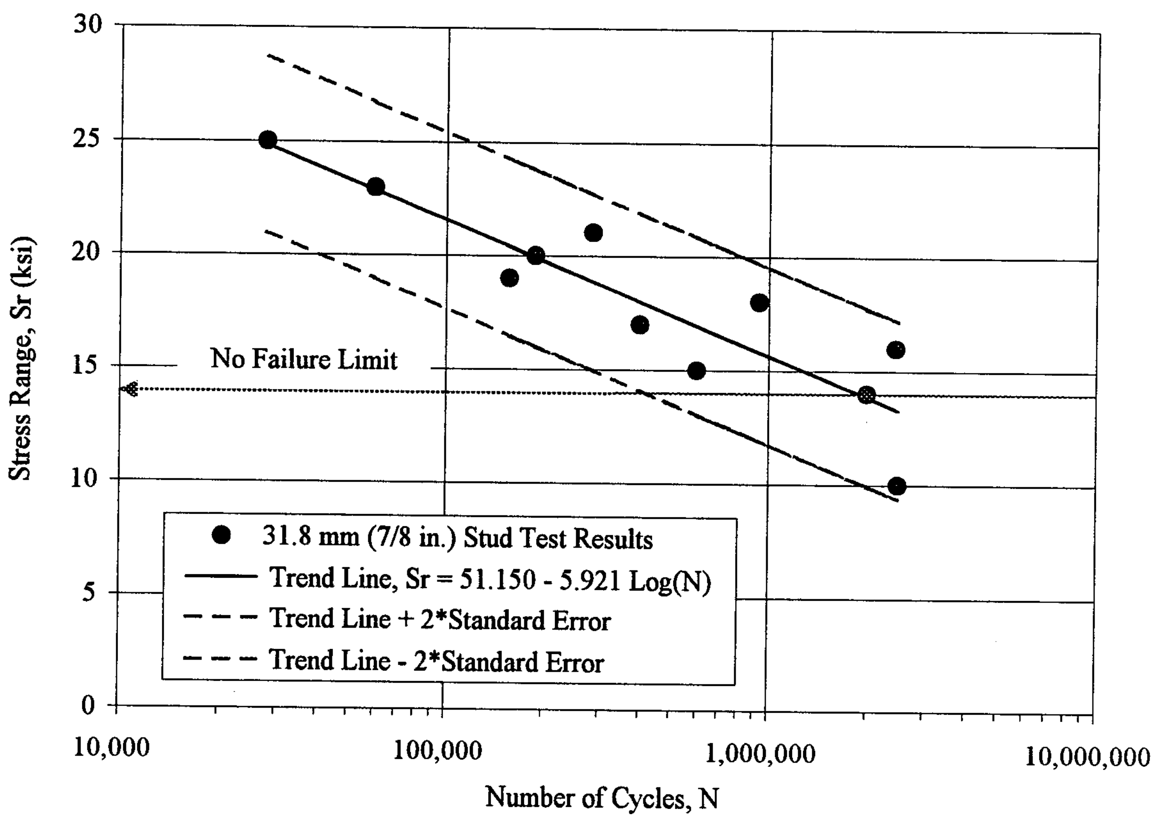


Figure 2.17 Test Results and Regression Analysis

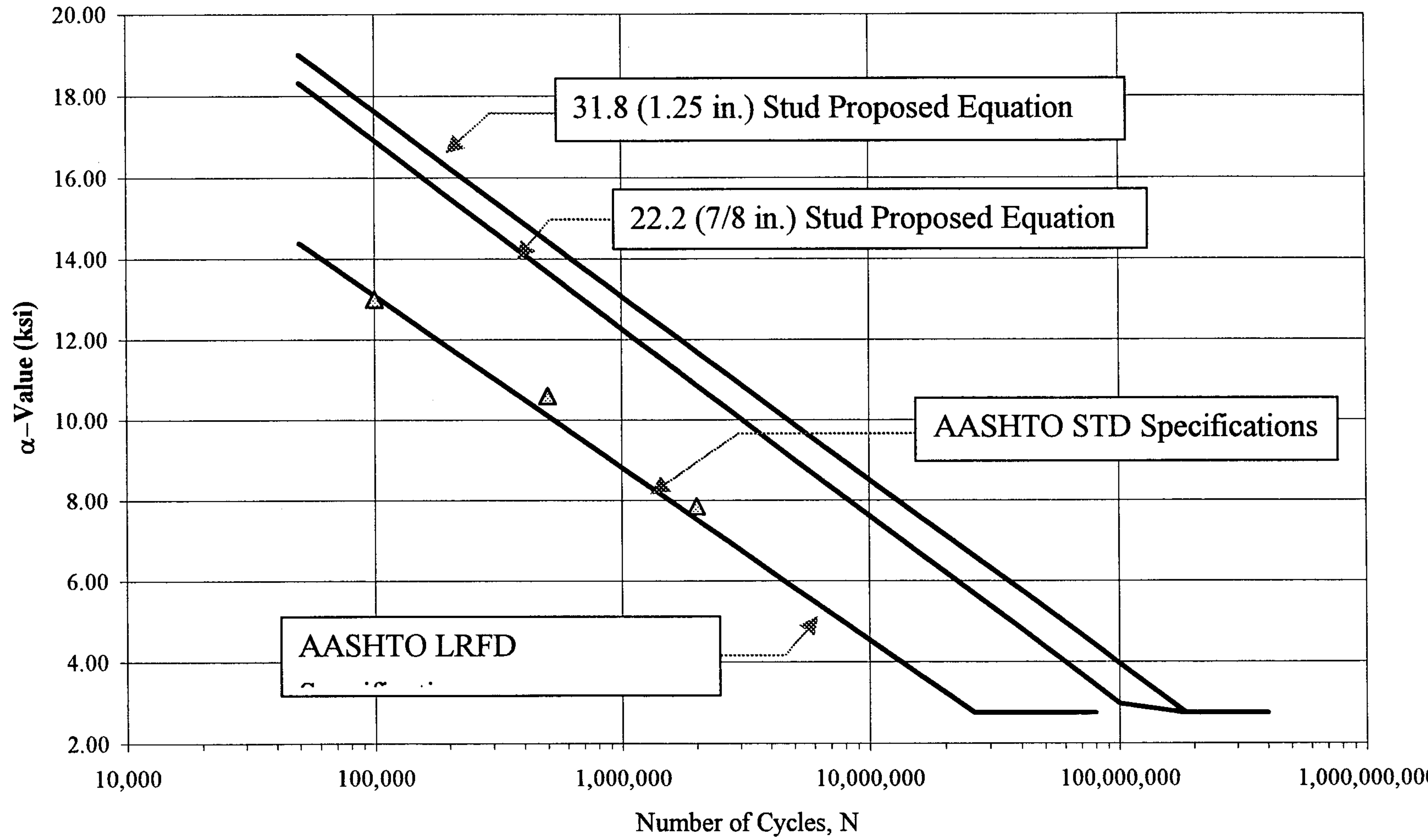


Figure 2.18 Comparison of the α-Values

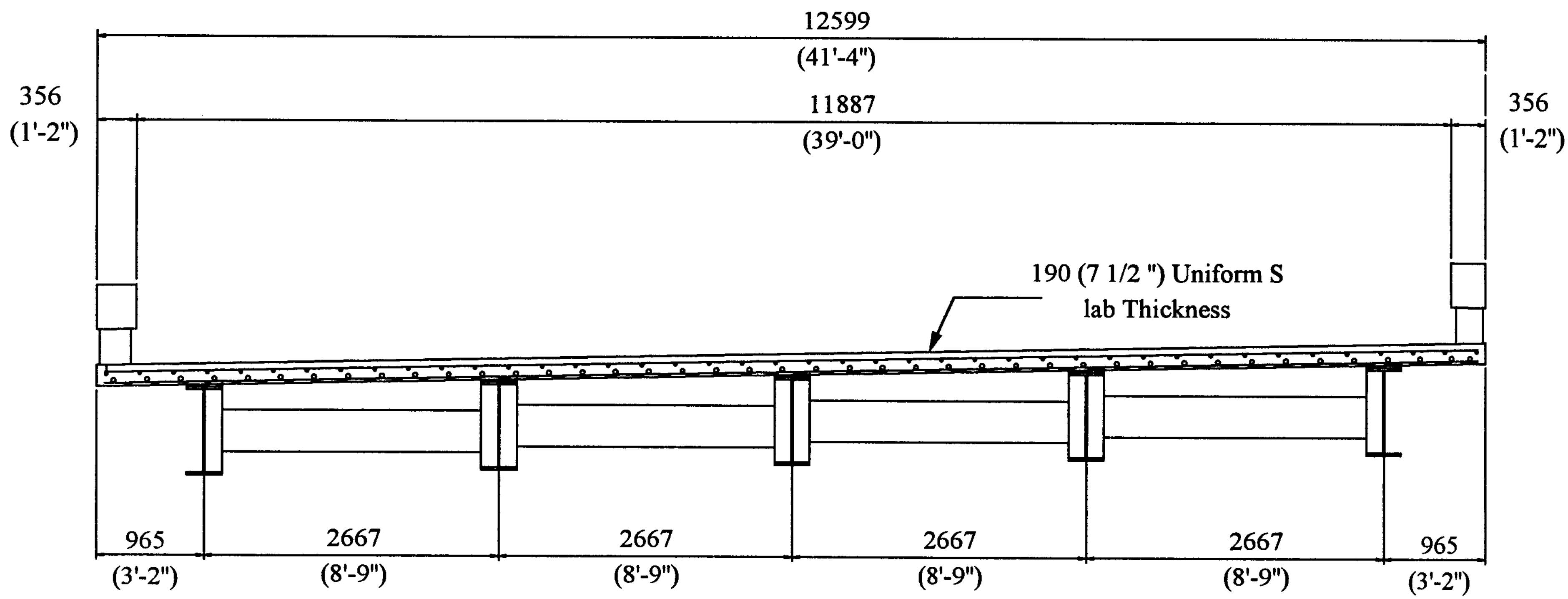
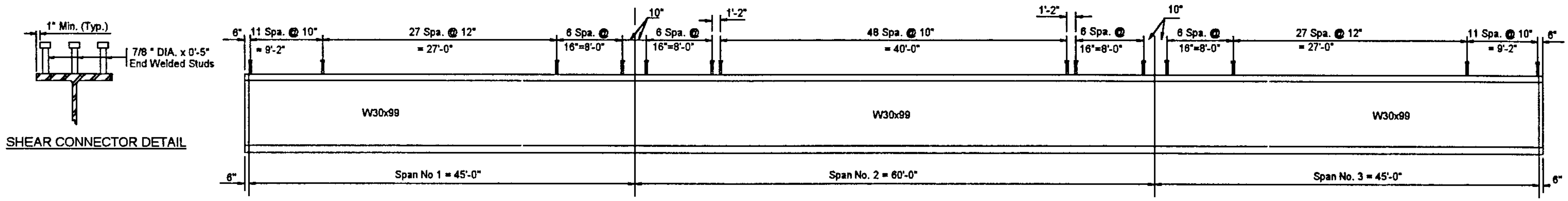
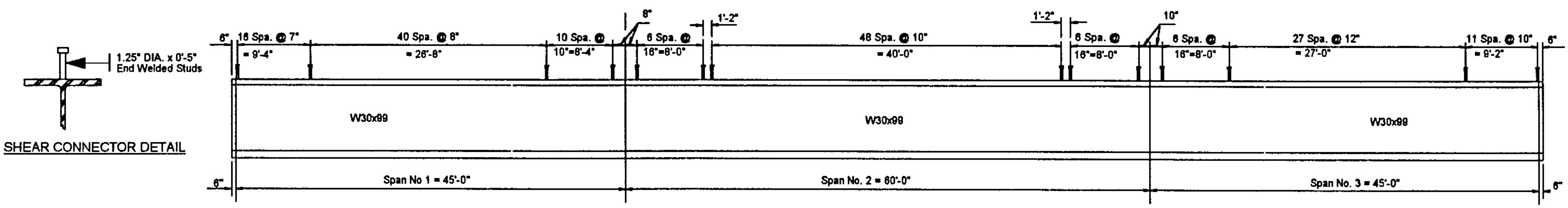


Figure 2.19 Typical Cross Section of the Demonstration Bridge



a) Preliminary Design with the 22.2 mm (7/8 in.) Stud



b) Final Design with the 31.8mm (1¼ in.) Stud

Figure 2.20 Preliminary and Final Stud Design for the Exterior South Span

1 in. = 25.4 mm

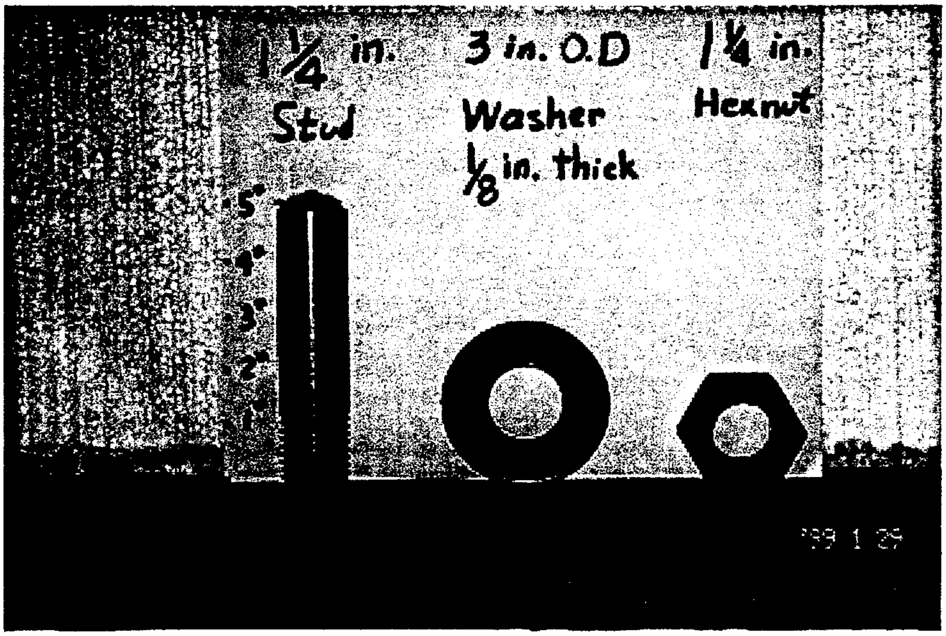


Figure 2.21 Shear Connector Parts

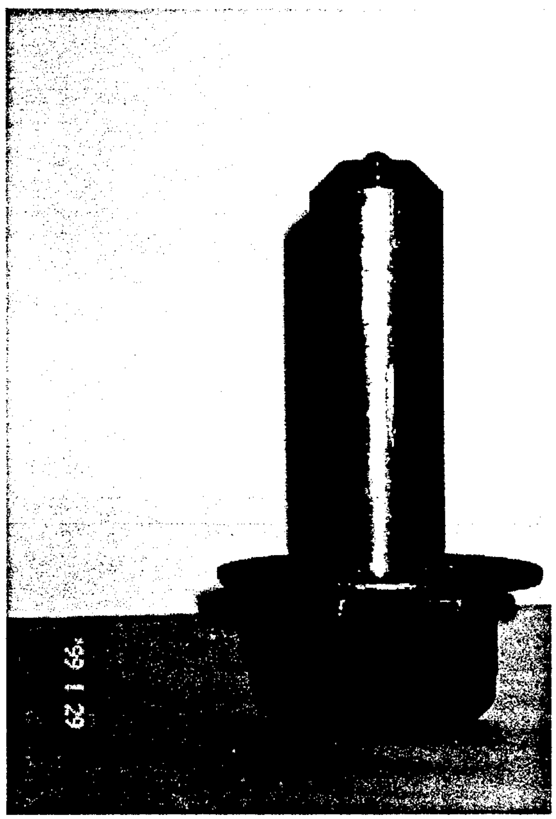


Figure 2.22 Shear Connector

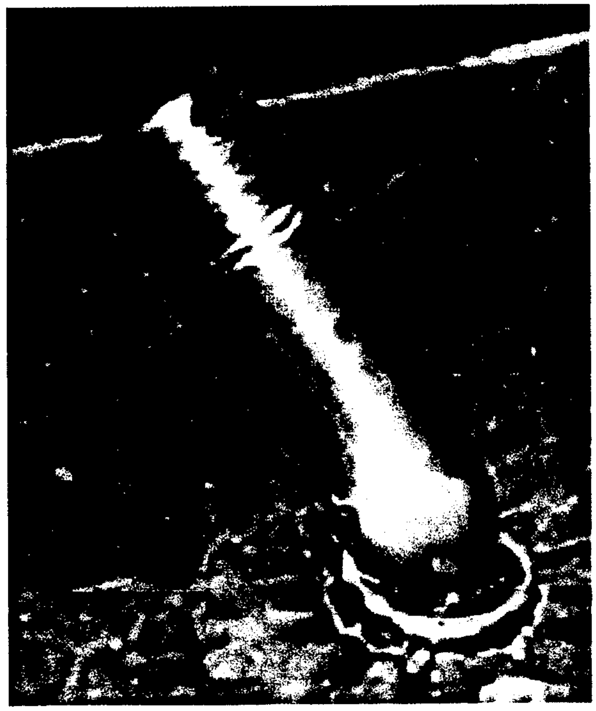


Figure 2.23 Quality Test by Bending the 31.8 mm (1¼ in.) Stud to 45 degrees

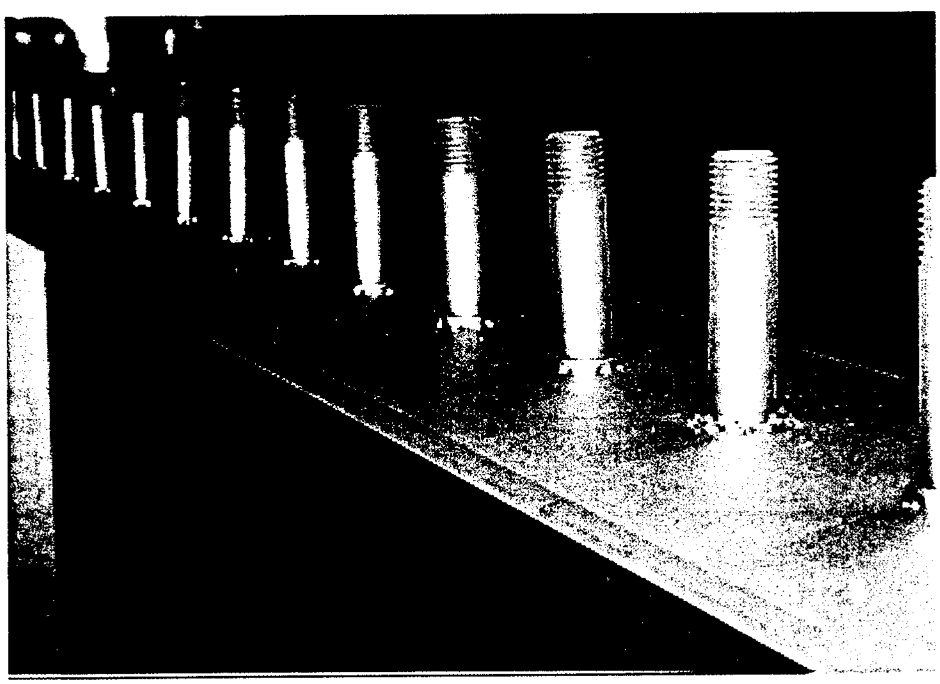


Figure 2.24 Large Studs after Blasting

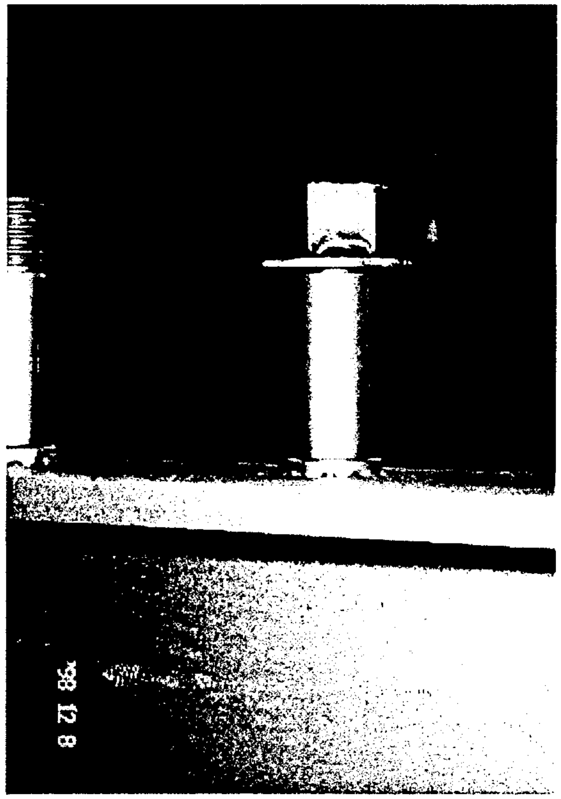


Figure 2.25 Complete Shear Connector

Figure 2.26 Steel Girders with the 31.8 mm (1¼ in.) Studs in Place on the South Span

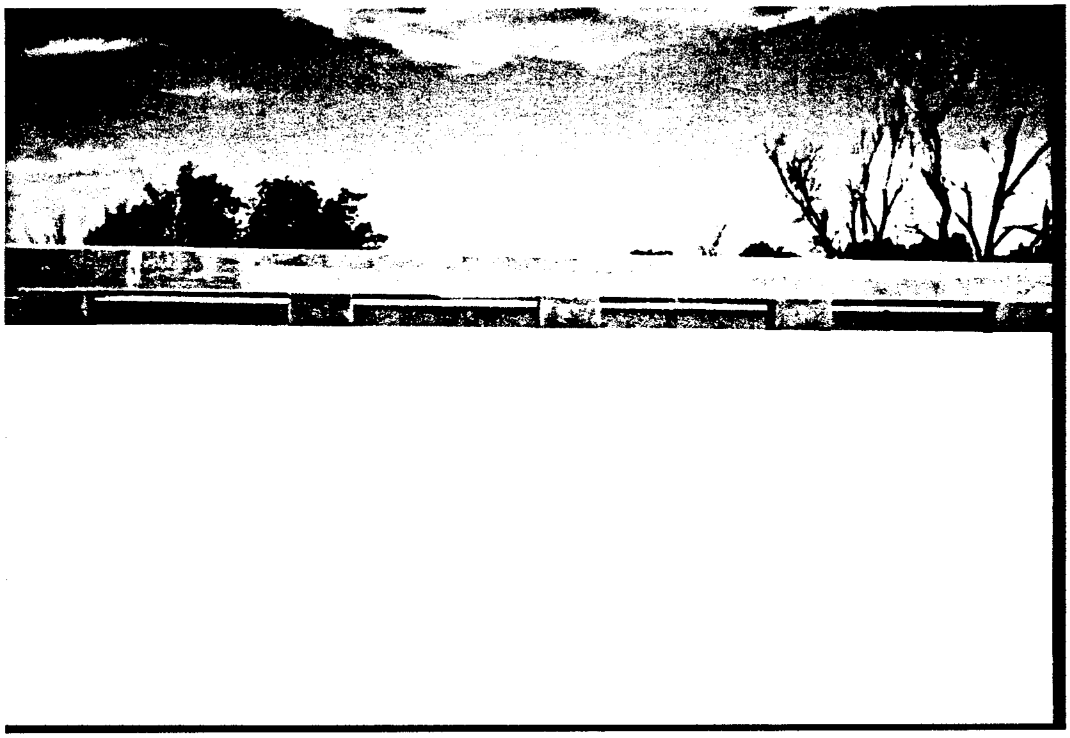


Figure 2.27 Top View of the Completed Deck

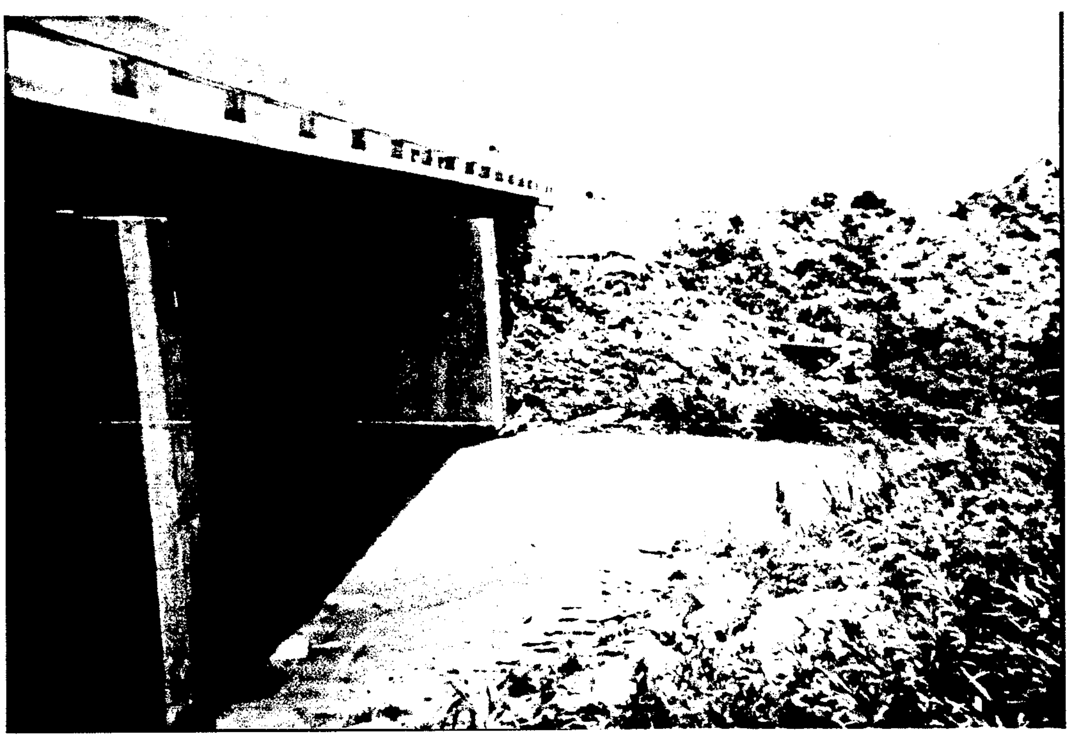


Figure 2.28 Side View of the Completed Bridge

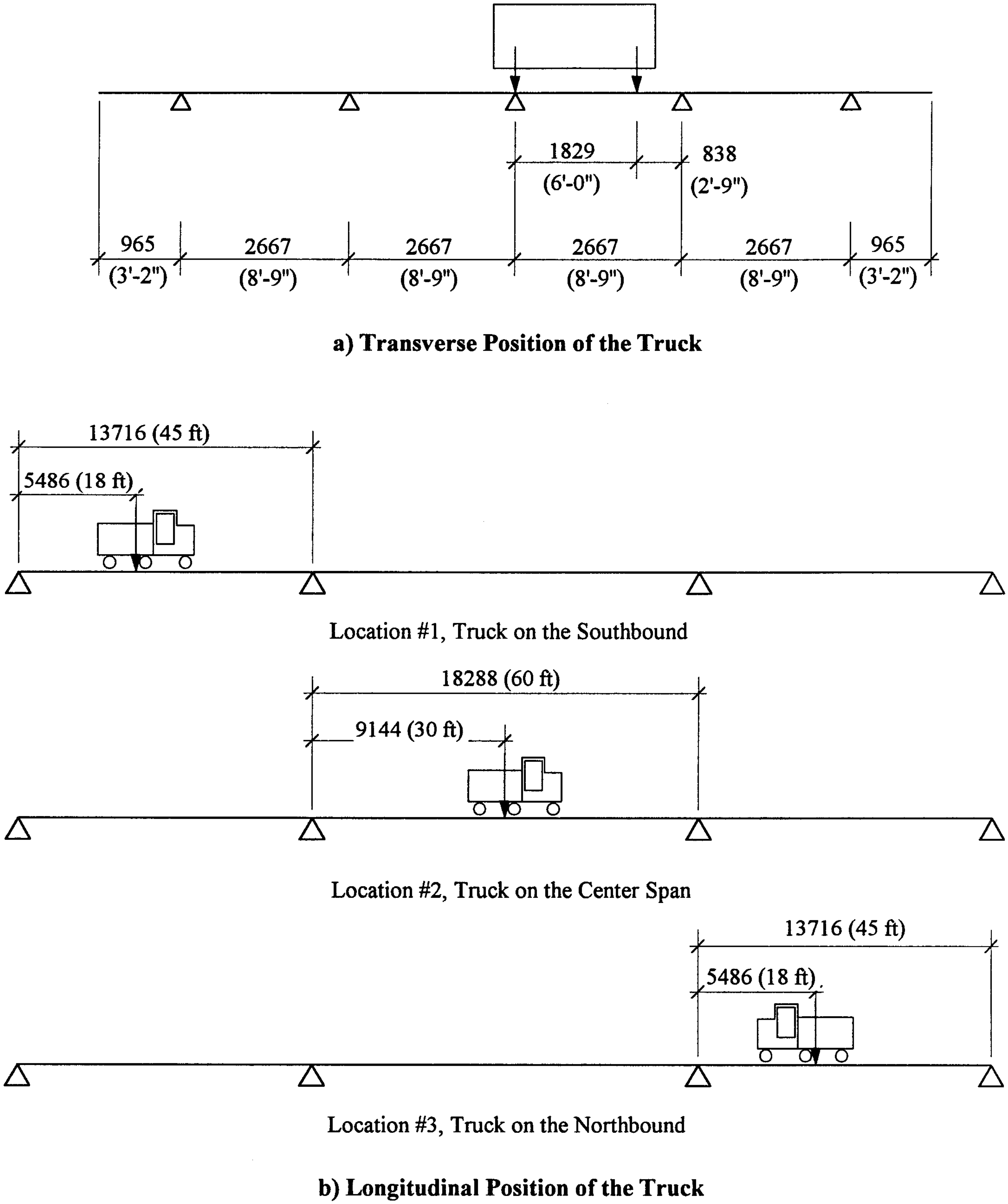


Figure 2.29 Deflection Measurement Arrangement

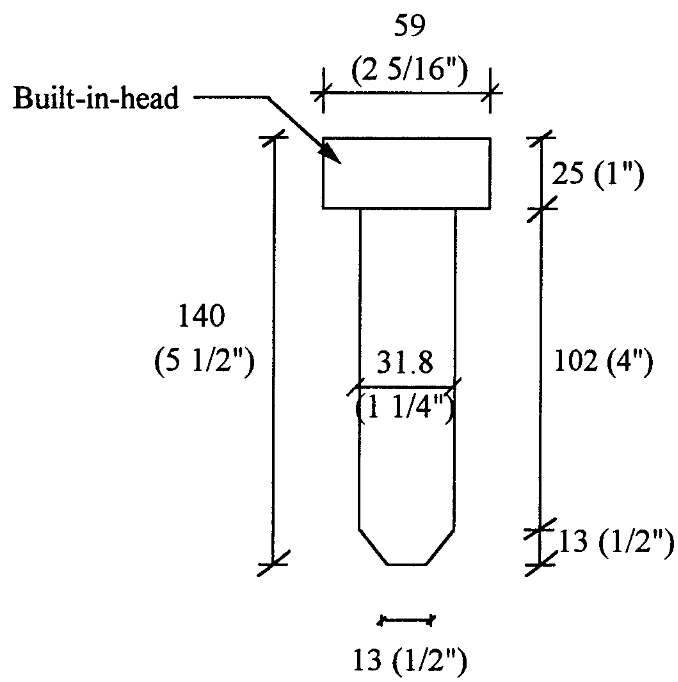


Figure 2.30 Dimensions of the Headed 31.8 mm (1 1/4 in.) Stud as Proposed by Continental Studwelding, Ltd.

CHAPTER 3

DEBONDED SHEAR KEY SYSTEM FOR CONCRETE COMPOSITE CONSTRUCTION

3.1 INTRODUCTION AND BACKGROUND

Nebraska Department of Roads (NDOR) uses a roughened interface system with shear connectors for composite concrete members for bridges. The shear connectors are provided by extending the vertical shear reinforcement of the girder outside the top flange. They have an L-shape and they are not epoxy coated, as shown in **Figure 3.1**. To prevent the thin top flange of the NU I-Girders from damage during deck removal, NDOR smoothes and debonds the outside 203.2 mm (8 in.) of the top flange by applying debonding agent [Bridge Division Policies and Procedure Manual (2000)]. Although this system has been used for a long time, it has the following drawbacks:

1. The shear connectors are susceptible to corrosion because they are not epoxy coated.
2. Debonding part of the top flange results in relatively large area of shear connectors because partial width of the wide top flange is used in the design.
3. Although the outside parts of the top flange are protected from damage during deck removal, the rest of the top flange is still at risk of damage because it is bonded with the deck.

For these reasons, there was a need to develop a new debonded interface system that optimizes the design for horizontal shear. This system uses the full width of the top flange and protects the top flange from damage during deck removal and protects the shear connectors from corrosion.

3.1.1 Debonded Shear Key Interface System

The idea of using a debonded concrete interface with shear connectors for composite concrete members was developed at the University of Nebraska in NCHRP Project 12-41 titled "Rapid Replacement of Bridge Decks" [Tadros and Biashya (1998)]. The debonded concrete interface consists of: (1) concrete shear keys provided on the top flange surface of the girder, (2) debonding sealant applied to the girder top flange surface, and (3) epoxy coated shear connectors of reinforcement bars placed at wide spacing extending from the girder web into the concrete deck slab. **Figure 3.2** shows a three-dimensional view of NU-girder with the proposed connection system and **Figures 3.3 and 3.4** show the shear key system of the NU-Girder that was used in the full-scale testing program [Tadros and Biashya (1998), Kamel (1996), Kakish (1997)]. Two types, Types I and II, of shear connectors were used in the full scale beam testing, as shown in **Figure 3.2**. In both types, a three-dimensional #5 bent bar was used. Shaping the shear keys by forming with steel forms was found to be the most efficient method for constructing the shear keys. The steel forms were attached to the top of the girder steel forms and the concrete was poured through the steel forms gaps, as shown in **Figure 3.5 and 3.6**.

Figure 3.7 gives the analytical model of the shear key mechanism. The angle of the shear key sides allows the two faces to slide across one another; hence, the connecting bar starts to engage in the mechanism. While sliding occurs, the two surfaces separate from each other causing tensile and shear stresses in the steel connector. To simplify the analysis, bending stresses in the connector are neglected, as the moment arm is very small. The system of forces acting on this model are: (1) the applied force P, (2) the tensile force in the connector, P_y, (3) the shear force in the connector P_{xy}, (4) the bearing force on the side of the shear key R, and (5) the friction force on the side of the shear key δR, where δ is the "local" coefficient of friction between the two surfaces. The two surfaces are assumed to be smooth.

3.1.2 Shear Key Dimensions

The debonded shear key system assumes that the shear keys have enough strength to resist the horizontal forces and allow sliding of the two concrete layers until failure occurs in the connector but not in the shear key. Two failure modes in the concrete shear key are expected: (1) bearing failure at the side of the shear keys, and (2) shear failure at the base plane of the shear key. Based on these failure modes, shear key dimensions can be determined as follows:

1) Bearing failure at the side of the shear keys:

$V_{uh} \leq \phi\ V_{nh}$

$v_{uh}\ (b_v\ S_{sk}) \leq \phi\ (0.85 f_c^{'})(t_{sk})(b_{sk} - t_{sk})$ Equation 3.1

$\phi = 0.7$ for bearing design

2) Shear failure at the base plane of the shear key (using the shear friction theory):

$V_{uh} \leq \phi\ V_{nh}$

$v_{uh}\ (b_v\ S_{sk}) \leq \phi\ (c\ A_{sk} + \mu\ A_{vf}\ f_y)$ Equation 3.2

$\phi = 0.9$ for shear design

Where (see **Figure 3.2** for illustration of various parameters):

A_{sk} = area of the shear key at base= $(b_{sk}\ w_{sk})$

A_{vf} = horizontal-shear reinforcement crossing the interface

b_{sk} = width of the shear key at base

b_v = width of the cross section at the contact surface being investigated for horizontal shear (interface width)

c = cohesion stress

f_c' = specified compressive strength at 28-days of the deck

f_y = specified minimum yield strength of reinforcing bars

t_{sk} = depth of the shear key

v_{uh} = factored horizontal shear stress at section due to composite loads

w_{sk} = length of the shear key at base

ϕ = strength reduction factor

μ = coefficient of friction

For concrete cast monolithically, μ=1.4, and c =1.03 MPa (0.15 ksi), as given by Section 5.8.4.2 in AASHTO LRFD Specifications (1998).

3.1.3 Design of Horizontal Shear Reinforcement

Testing proved that beams designed with the debonded shear keys interface system, with sufficient shear connectors that satisfy horizontal shear, performed comparably with similar beams designed with conventional roughened system, both in flexure and shear. Also, testing results showed that the shear friction theory could be safely applied to the design, using a friction coefficient "μ" = 1.0 and a cohesion stress "c" = zero, as follows:

$$V_{uh} \leq \phi \, V_{nh}$$

$$v_{uh} \, (b_v \, S_{sk}) \leq \phi \, (c \, A_{sk} + \mu \, A_{vf} \, f_y)$$

$$v_{uh} \, (b_v \, S_{sk}) \leq 0.9 \, (0 + 1.0 A_{vf} \, f_y)$$

$$A_{vf} = \frac{v_{uh}(b_v S_{sk})}{0.9 f_y} \qquad \text{Equation 3.3}$$

Note that a cohesion stress "c" = zero is used because at the interface there is no bond between the girder top flange and the concrete deck (debonded system). Theoretical analysis of the debonded shear key system and the verification testing program can be found in Kamel (1996), Kakish (1997), and Tadros and Biashya (1998).

3.2 RESEARCH OBJECTIVES

Objectives of this task of the research project were to:

1. Finalize the shear key details for the NU I-Girders.
2. Develop design criteria and specifications for the debonded shear key system.
3. Implement the debonded shear key system on a demonstration bridge and document its installation.
4. Monitor the structural behavior of the demonstration bridge.
5. Prepare an economy analysis of using the debonded shear key system.

3.3 FINALIZATION OF THE DEBONDED SHEAR KEY SYSTEM

3.3.1 Debonded Shear Key System Details

The research team held many meetings with NDOR bridge engineers to present the debonded shear key system, its details, and the testing results of the NCHRP Project 12-41. NDOR bridge engineers had the following comments:

1. Providing the shear keys as projecting outside the top flange may create problems in the field when using stay-in-place panels such as the conventional stay-in-place panels or the NUDECK system [Badie et al. (1998)].

2. The three-dimensional shear connectors, Type I or II, are relatively expensive and may significantly raise the girder cost.
3. The three-dimensional shear connectors are very wide such that they will require a large amount of grouting when using stay-in-place panels such as the conventional stay-in-place panels or the NUDECK system [Badie et al. (1998)].

Based on these comments, UNL researchers modified the debonded shear key details as follows:

1. The shear keys are formed as dips in the top flange. According to the NU I-Girder reinforcement standard details, a shear key as deep as19.1 mm (¾ in.) can be created in the top flange. Although providing the shear keys this way will affect the geometric properties of the girder, the reduction will be very small and will not affect the stress of the final composite section because the dips will be filled with the deck concrete.
2. The three-dimensional shear connector is replaced with #16 (#5) U-shape bars that are imbedded in the web of the girder and extending outside the top flange.

Figure 3.7 gives the final details of the shear key system. Note that the dimensions of the shear keys are similar to those used in NCHRP Project 12-41 [Tadros and Baishya (1998)]. These modified details were approved by NDOR bridge engineers.

3.3.2 Composite Loads

An important issue that was discussed between NDOR bridge engineers and UNL researchers was which loads should be used to calculate the applied horizontal stress "v_{uh}" at the interface. Neither the AASHTO Standard Specifications (1996) nor the LRFD Specifications

(1998) give guidance in this regard. While most designers would use all loads to calculate "v_{uh}", a strong case can be made for excluding the own weight of the girder and the weight of the slab since they are present prior to composite action taking effect. This approach can be explained as follows:

Figure 3.9a shows a beam subjected to its own weight only. When a topping slab is cast over the beam, **Figure 3.9b**, the beam stresses are increased due to the dead weight of the wet concrete. However, even after the slab concrete is hardened, the stresses in the slab are theoretically zero since that slab weight was provided only to the non-composite section. Therefore, no horizontal shear stresses exist at the interface due to either the beam or the slab weight. After the slab hardens, the beam and the slab form a composite member, and any load applied after that will be resisted by the composite section and produce interface horizontal shear stresses, as shown in **Figure 3.9c**. These loads are called composite loads. In general case of bridge design, composite loads consist of superimposed dead loads and live loads. Superimposed dead loads include barriers and wearing surface. Live loads include truck, lane, or pedestrian load.

Some designers and agencies use only the composite loads to calculate "v_{uh}", such as Illinois Department of Transportation [Bridge Manual (1999)], which has been using this approach for a long time without any reported problems.

3.3.3 Factored Horizontal Shear Stresses for Composite Sections

Theoretical calculation of the factored shearing stress at the interface of a composite member is not simple because the section does not behave as a linear elastic material near ultimate stage. If it did, the shear stress could be calculated as follows:

$$v_{uh} = \frac{V_u Q}{I b_v}$$ Equation 3.4

Where:

b_v = width of the interface

I = moment of inertia of the composite section

Q = first moment of the area above the interface

V_u = ultimate vertical shear force at a section

Using Equation 3.4 for reinforced concrete section yields questionable results. This is because, at ultimate stage, the material is no longer elastic and the concrete may be cracked at the section being considered. Also, the composite section consists of two different types of concrete with different properties. Loov and Patnaik (1994) determined that Equation 3.4 might yield adequate results if the terms "I" and "Q" are determined based on a cracked transformed section. In this case, the section would be transformed using the slab-to-beam modular ratio used in flexural design by the allowable stress method. However, this approach is relatively complicated because it uses both service and ultimate limit states in the calculations.

The ACI 318 Code (1999) and the AASHTO Standard Specifications (1996) implies the following equation for calculating "v_{uh}":

$$v_{uh} = \frac{V_u}{b_v d}$$ Equation 3.5

Where:

d = depth of steel reinforcement

Using equilibrium of forces, Kamel (1996) showed that "v_{uh}" can be determined by using the following equation:

$$v_{uh} = \frac{V_u}{b_v\, d_v}$$ Equation 3.6

Where:

d_v = distance between the tension and compression resultant stresses in the section

$= (d - a/2)$

a = depth of compression stress block

It is clear that the ACI 318 Code (1999) and the AASHTO Standard Specifications (1996) simplified Equation 3.6 by eliminating the "a/2". However, this makes both of them under estimate the applied horizontal shear stress at the interface.

The AASHTO LRFD Specifications (1998) gives no guidance for computing horizontal shear stresses due to factored load.

UNL researchers recommend Equation 3.6 be used in calculating the factored horizontal shear stresses for composite members with the new debonded shear key interface system or with the conventional roughened interface system. The same approach has been recommended by the Precast/Prestressed Concrete Institute Bridge Design Manual PCI-BDM (1997).

3.3.4 Maximum Reinforcement Limit

Literature review showed that a number of different maximum reinforcement limits have been recommended. Among them is the limit that is given by Section 5.8.4.1 of the LRFD Specifications [AASHTO LRFD (1998)], which gives the following limits:

$$A_{vf}\, f_y \leq 0.2\ f_c^{'}(b_v\, S_{sk})$$ Equation 3.7

Where f'_c is the strength of the beam or the slab whichever is smaller and where f'_c may be not taken greater than 28 MPa (4.0 ksi). This limit that was developed in the late 1970s is very conservative. The authors recommend using the limit given by Loov (1994) as follows:

$$A_{vf} f_y \le 0.25 \, f'_c (b_v S_{sk}) \qquad \text{Equation 3.8 (English units)}$$

3.4 DESIGN CRITERIA AND SPECIFICATIONS FOR THE DEBONDED SHEAR KEY INTERFACE SYSTEM

Based on the finalized details and issues discussed in the previous sections of this chapter, UNL researchers developed the following design procedure for use in the design of the debonded shear key interface system.

Using shear key dimensions shown in **Figure 3.8** and a specified concrete strength of the deck of 28 MPa (4.0 psi):

1. Using a continuous beam analysis program, calculate the unfactored (service load) vertical shear forces per girder due to composite dead loads (overlay, future wearing surface, barriers, etc.) and live loads (V_{cd} and V_{L+I}, respectively).
2. Determine the factored vertical shear forces due to composite dead and live loads, V_u, according to the adopted specifications.
3. Determine the factored horizontal shear stress due to composite dead and live loads using

$$v_{uh} = \frac{V_u}{b_v d_v} \qquad \text{Equation 3.6}$$

4. Check that: $v_{uh} \le 0.945$ MPa (0.137 ksi)

This limit is developed based on Equation 3.1, to protect the shear key against bearing failure, using the shear key dimensions given in **Figure 3.8** and a specified concrete strength of the deck, 28 MPa (4 ksi).

If v_{uh} exceeds this limit, the specified concrete strength of the deck, 28 MPa (4 ksi), should be increased and/or the dimensions of the shear key should be changed. This can be done by solving simultaneously Equation 3.1 and 3.2.

$$v_{uh}\,(b_v\,S_{sk}) \le \phi\,(0.85 f_c^{'})(t_{sk})(b_{sk} - t_{sk}),\ \phi = 0.7 \text{ for bearing design} \qquad \text{Equation 3.1}$$

$$v_{uh}\,(b_v\,S_{sk}) \le \phi\,(c\,A_{sk} + \mu\,A_{vf}\,f_y), \qquad \text{Equation 3.2}$$

$\phi = 0.9$ for shear design, c = 1.03 MPa (0.15 ksi) and $\mu = 1.4$

Since these equations have four variables (t_{sk}, b_{sk}, w_{sk}, and $f_c^{'}$), two variables have to be chosen in advance by the designer and then the remaining two variables can be calculated.

5. Determine required horizontal-shear reinforcement, A_{vf}:

$$A_{vf} = \frac{v_{uh}\left(b_v S_{sk}\right)}{0.9 f_y} \qquad \text{Equation 3.3}$$

6. Check maximum reinforcement limits:

$$A_{vf}\,f_y \le 0.25\ f_c^{'}\,(b_v\,S_{sk}) \qquad \text{Equation 3.8}$$

The design steps can be programmed into a spreadsheet. A detailed example illustrating this procedure is given in the following section.

3.5 DEMONSTRATION PROJECT

Nebraska Department of Roads (NDOR) assigned a simple span bridge in Nebraska on which to implement the debonded shear key system. The bridge is on the new Highway 64, Waterloo Northwest, Douglas County, NE, Structure No. S275-1584L&R.

3.5.1 Project Description

The project consisted of two identical simple-span bridges (Northbound and Southbound Structures) of 39.01 m (128 ft). Cross-section of both bridges consisted of five 1600 NU-girders spaced at 2.5 m (8 ft –2½ in.) supporting a 190 mm (7.5 in.) thick composite cast-in-place slab. Total width of the bridge was 12.4 m (40 ft –8 in.), as shown in **Figure 3.10**. The conventional roughened interface system was used on the Northbound Structure, while the new debonded shear key system was used on the Southbound Structure.

Preliminary design of the horizontal shear reinforcement of the composite action included 2-D18 at spacing ranging from 50 to 300 mm (1.97 to 11.81 in.). NDOR agreed to implement the new debonded shear key system on the Southbound Structure, as shown in **Figure 3.11**, while using the conventional roughened interface system on the Northbound Structure, as shown in **Figure 3.12**. This arrangement was chosen to give NDOR the opportunity to compare the structural performance between the conventional and new interface systems.

3.5.2 Design Calculations of the Debonded Shear Key System for the Southbound Structure

Design criteria and specifications, as outlined in Section 3.3, were used in the design of the debonded shear key system for the Southbound Structure. Steps of the design are illustrated in **Table 3.1**. Dimensions of the shear key and details of reinforcement are shown in **Figures 3.12 and 3.13**. Using the debonded shear key system resulted in 2-U #16 (#5) bars at spacing ranging from 300 to 600 mm (11.81 to 23.62 in.). To illustrate the design steps given in Section 3.3 of this chapter, design calculations at Section $x/L = 0.1$ are shown below:

Step #1: Unfactored vertical shear due to composite dead and live loads:

V_{cd} = 60.5 kN (13.6 kips)

V_{L+I} = 290 kN (65.2 kips)

Step #2: Factored horizontal shear stress due to composite dead and live loads:

$V_u = 1.3\ [V_{cd} + (5/3)\ V_{L+I}]$ = 707 kN (159 kips)

Step #3: Factored horizontal shear stress due to composite dead and live loads:

d = 1527 mm (60.12 in.)

a = 177 mm (7.0 in.)

$d_v = d - a/2$ = 1438 mm (56.62 in.)

b_v = 1225 mm (48.2 in.)

$$v_{uh} = \frac{V_u}{b_v d_v} = 0.400 \text{ MPa } (0.058 \text{ ksi})$$

Step #4: Check that: $v_{uh} \leq 0.95$ MPa (0.137 ksi) OK

Step #5: Required horizontal-shear reinforcement, A_{vf}:

S_{sk} = 300 mm (11.81 in.)

f_y = 414 MPa (60 ksi)

$$A_{vf} = \frac{v_{uh}(b_v S_{sk})}{0.9 f_y} = 395 \text{ mm}^2 \ (0.61 \text{ in}^2)$$

If 2-U #16 (#5) bars are used, thus,

A_{vf} (provided) = 4 x 0.31 = 800 mm^2 (1.24 in^2)

Required spacing of the 2-U #16 (#5) bars = $\dfrac{800(1.24)}{395(0.61)} = 2.03$

Use 2-U #16 (#5) bars at alternate shear keys

Step #6: Check maximum reinforcement limits:

$A_{vf} f_y \leq 0.25\ f_c^{'}(b_v S_{sk})$

$(0.5x1.24)(60) = 165.5$ kN (37.2 kips)

$0.25(4.0)(48.2x11.81) = 2532$ kN (569.2 kips) OK

3.5.3 Girder Production

The precast concrete girders were produced by Concrete Industries, Inc., Lincoln, NE. Although the research team recommended the use of steel forms to form the shear keys, the precaster used wood forms. Production of the girder with the shear keys went smoothly without any problems. The top surface of the exposed shear key was steel troweled to a smooth finish. After the girders were removed from the prestressing bed, the top flange was sprayed with bond breaker (a sealant) to break the bond between the girder top flange and the concrete deck. **Figures 3.14 to 3.17** show the girders with the shear keys.

NDOR bridge engineers and UNL researchers inspected the girders at the precast yard, where the following recommendations were issued to be considered in future projects:

1. The precasters placed the 2-#16 (#5) U-bars back-to-back with no spacing between them, as shown in **Figures 3.14 and 3.15**, to avoid conflict with the draped strands at girder ends. This would not allow the deck concrete to fully surround the shear connectors, and it was expected that this would reduce the resistance that should be produced by these shear connectors. Thus, UNL researchers recommended that a crossbar should be passed through the U-bars to provide mechanical anchorage to the connector. This could be done by passing the deck slab reinforcement through the ties or by using short pieces of bars. To determine

the size and the length of the short bars, UNL researchers used the anchorage requirements that are used with headed studs as follows:

The ratio between the area of the stud head (A_2) and the area of the stud stem (A_1) for several products is as follows:

- 22.2 mm (7/8 in.) diameter stud: $(A_2/A_1) = (1.25^2)/(7/8^2) = 2.04$
- 31.8 mm (1¼ in.) diameter stud with 76mm (3 in.) diameter washer and nut:

$$(A_2/A_1) = (3^2)/(1.25^2) = 5.76$$

- Decon STUDRAIL: $(A_2/A_1) = 10$

Using an average ratio of 6, the required bearing (stud head) area for 2 #5 U-ties is:

$$A_2 = (4x0.31)(6) = 4800 \text{ mm}^2 \, (7.44 \text{ in}^2)$$

This area can be provided by using any of these alternatives:

- 254-mm (10-in.) long, #19 (#6) epoxy coated bar or 127-mm (5-in.) long, 2 #19 (#6) epoxy coated bars; $A_2 = 10(6/8) = 7.5 \text{ in}^2$
- 203-mm (8-in.) long, #25 (#8) epoxy coated bar or 102-mm (4-in.) long, 2 #25 (#8) epoxy coated bars; $A_2 = 8(8/8) = 8.0 \text{ in}^2$
- 76-mm (3-in.) long, 2.5 in. outside diameter, 3.18 mm (1/8 in.) wall thickness galvanized tube; $A_2 = 3x2.5 = 7.5 \text{ in}^2$

The project contractor chose to use 254-mm (10-in.) long, #19 (#6) epoxy coated bar.

It is recommended that in future projects the shear connectors be placed outside the draped strands to create adequate concrete cover around the connectors and help their anchorage with the deck concrete.

2. Using wood in forming for the shear keys resulted in holes on the girder top surface, as shown in **Figure 3.16**. Also, building the wood forms reduced the construction speed. It is

recommended that in future projects steel forms be used rather than wood forms to increase construction speed and to enhance the top girder top surface smoothness.

3. At the girder end, where the top flange is eliminated, the precasters used the conventional non-epoxy coated L-shape shear connectors. Also, the shear keys were eliminated for a distance of 508 mm (20 in.) at girder ends to accommodate the lifting hooks, as shown in **Figure 3.17**. It is recommended that for future projects that the shear keys be provided to the end part of the top flange and that the lifting hooks be considered as the shear connectors if they conflict with the shear keys. Also, it is recommended that the same U-shape shear connectors be used through out the girder span.

3.5.4 Construction of the Demonstration Project

Construction of the demonstration project started early in the summer of 1998 and was completed by end of summer 1999. **Figures 3.18 to 3.23** show different views of the Northbound and Southbound structures during construction. The contractor commented that using the U-shape shear connectors rather than the L-shape conventional connectors enhanced the safety of the construction workers and left them with more space on the girder top surface.

3.5.5 Deflection Measurements of the Demonstration Project

The construction of the bridge was completed in August 1999. However, the bridge is not open for public traffic because the construction of the new Highway 64 is not completed yet. The deflection measurements of the bridge were taken on November 30, 1999. A three-axle truck was used. The truck was positioned transversely such that one-line of wheels was set directly over the center girder, as shown in **Figure 3.24a**. The truck was positioned longitudinally such that the center axle of the truck was set exactly over the mid-span section, as shown in **Figure 3.24b**. The weight of each axle was recorded before and after taking the measurements at two different weighing stations, see **Figure 3.24b**.

Deflection measurement was taken for the Southbound Structure, which has the debonded shear key system, and the Northbound Structure, which has the conventional roughened shear key system, by two methods as follows:

1. The transit was set on the west hill behind the west abutment and a fixed point was chosen as a reference. Using a metric scale attached to the bottom surface of the center girder, the first reading was taken with the truck positioned on the bridge. Then, the truck was pulled off the bridge and a second reading was taken. The difference between the two readings gave the deflection of the center girder.
2. A right angle triangle was set and the angle of the triangle and the distance from the transit to the bottom surface of the center girder was measured using laser transit. Deflection measurement was taken first with the truck positioned on the bridge and then after the truck was pulled off the bridge.

Recorded deflection readings were as follows:

	Southbound Structure (Debonded shear key system)	Northbound Structure (Conventional roughened shear key system)
Method I	2 mm	1 mm
Method II	1 mm	3 mm

To compare field deflection measurements with theory, the lever rule was used to calculate the distribution factor for the center girder. From **Figure 3.24a**, reaction at the center girder = $0.5P + 0.5P(\frac{2.2}{8.2}) = (0.634)P$. Thus, distribution factor for the moment = 0.634

Composite properties of the center girder are as follows:

Cross section area = 8.819×10^5 mm^2 (1367 in^2)

Inertia = 4.01899×10^{11} mm^4 (965567 in^4)

Modulus of elasticity = $33(150)^{1.5}\sqrt{8{,}000}/1000$ = 37,385 MPa (5,422 ksi)

Minimum specified concrete strength of the precast girders after 28 days $f_c^{'}$ = 55 MPa (8 ksi)

Using continuous beam analysis software, deflection at mid span section = 11.05 mm (0.435 in.)

From the deflection measurements, the following conclusions can be drawn:

1. Field deflection measurement of the Southbound Structure, built with the debonded shear key system, is almost the same as that of the Northbound Structure, built with the conventional roughened shear key system. This means that the new shear key system has no detrimental effect on the composite action of the bridge.
2. Field deflection measurements are very small compared to the deflection calculated by theory, which means that the actual structure is stiffer than that considered in the analysis. This may be due to the fact that the concrete girder may have higher strength than the minimum specified value and/or the approximation used in calculating the distribution factor for the truck load.

Visual inspection of the deck top surface of both structures showed neither signs of distress nor difference in the structural behavior.

3.6 COST ANALYSIS

Using the dimensions of the demonstration bridge, the expected incremental production cost of girder due to the addition of the debonded shear key system can be summarized as follows:

1. Extra cost due to use of #5 (#16) U-bars (assuming 35 cents per pound for epoxy coated bars) = $126.00 per girder
2. Reduction in weight of the vertical shear reinforcement due to it being terminated at top flange (assuming 35 cents per pound for non-epoxy coated welded wire reinforcement) = $86.00 per girder
3. Sprayed debonding agent (assuming 10 cents per square ft) = $26.00
4. Troweling the spacing between shear key forms may be assumed equal to roughening the entire top flange in conventional production.

- Net incremental cost = 126 – 86 + 26 = $66.00 per girder

  = ($66.00 x 5 girders) / (128 ft x 40.75 ft) = $0.063 per sq. ft

However, actual incremental cost of the Southbound Structure girders was higher than the above figure because wood forms were used to produce the shear keys. This resulted in added cost of $305.00 per girder for material and labor used in making the forms, $437.00 per girder for extra production labor due to installing and removing the wood forms, and $88.00 per girder for extra engineering cost.

The authors believe that when the system becomes the standard practice and steel forms are used in future projects, the extra cost due to production, installing, and removing of the forms would be eliminated. Also, with repeated usage of this system, the precaster will not incur extra engineering cost. It is also important to note that the long-term saving in time and labor associated with future deck removal due to the use of debonded interface system, will compensate for any incremental cost.

3.7 CONCLUSIONS

The new debonded shear key system for concrete composite members has been successfully implemented in bridges. Results obtained from the extensive laboratory testing [Tadros & Baishya (1998)] and the field implementation of this research project, have proven that the new system is a competitive replacement for the conventional roughened interface system. The new system optimizes the design of the NU Girder design for horizontal shear, improves the quality of the deck by using epoxy coated shear connectors, minimizes the damage that may occur to the top girder flange during deck removal, and improves deck removal speed.

Table 3.1 Design Calculations of the Debonded Shear Key System for the Southbound Structure

Step	x/L	0.0	T.S*	H/2	0.1	0.2	0.3	0.4	0.5	Notes
(1)	V_{cd}, kN (kips)	74.7 (16.8)	73.0 (16.4)	71.3 (16.0)	60.5 (13.6)	45.4 (10.2)	30.2 (6.8)	15.1 (3.4)	0 (0)	From continuous beam analysis program
	V_{L+I}, kN (kips)	332.3 (74.7)	328.3 (73.8)	324.4 (72.9)	290.0 (65.2)	265.5 (59.7)	231.3 (52.0)	196.5 (44.2)	161.5 (36.3)	
(2)	V_u, kN (kips)	817.1 (183.7)	806.1 (181.2)	795.2 (178.9)	707.0 (159.0)	634.3 (142.6)	540.5 (121.5)	445.5 (100.2)	349.9 (78.5)	AASHTO STD Specifications $V_u = 1.3\ [V_{cd} + (5/3)\ V_{L+I}]$
(3)	d, mm (in.)	1472 (57.95)	1479 (58.23)	1485 (58.46)	1527 (60.12)	1584 (62.36)	1640 (64.57)	1697 (66.81)	1697 (66.81)	Depth of the prestressed reinforcement
	a, mm (in.)	0 (0)	56 (2.21)	93 (3.66)	177 (7.0)	177 (7.0)	177 (7.0)	177 (7.0)	177 (7.0)	Depth of the Compression stress block (from flexural anlysis)
	$d_v = (d - a/2)$, mm (in.)	1472 (58.0)	1451 (57.13)	1439 (56.63)	1438 (56.62)	1496 (58.90)	1551 (61.07)	1608 (63.31)	1608 (63.31)	
	v_{uh}, MPa (ksi)	0.455 (0.066)	0.455 (0.066)	0.448 (0.065)	0.400 (0.058)	0.345 (0.050)	0.283 (0.041)	0.228 (0.033)	0.179 (0.026)	Equation 3.6
(4)	$v_{uh} \le 0.95$ MPa $\le (0.130$ ksi)	OK	OK	OK	OK	OK	OK	OK	OK	
(5)	Required A_{vf}, mm^2/S_{sk} (in^2/S_{sk})	452 (0.70)	452 (0.70)	445 (0.69)	395 (0.61)	342 (0.53)	277 (0.43)	226 (0.35)	174 (0.27)	Equation 3.3
	Provided A_{vf} = 2-#5 (#16) U bars	800 (1.24)	800 (1.24)	800 (1.24)	800 (1.24)	800 (1.24)	800 (1.24)	800 (1.24)	800 (1.24)	
	Provide 2-#5 (#16) U bars at	1	1	1	2	2	2	3	4	shear keys
	Provided Spacing in (mm^2)	300 (11.81)	300 (11.81)	300 (11.81)	600 (23.62)	600 (23.62)	600 (23.62)	600 (23.62)	600 (23.62)	
(6)	$A_{vf}\, f_y \le 0.25\, f'_c\, A_{cv}$	OK	OK	OK	OK	OK	OK	OK	OK	Equations 3.8

* Transfer section of prestressed reinforcement

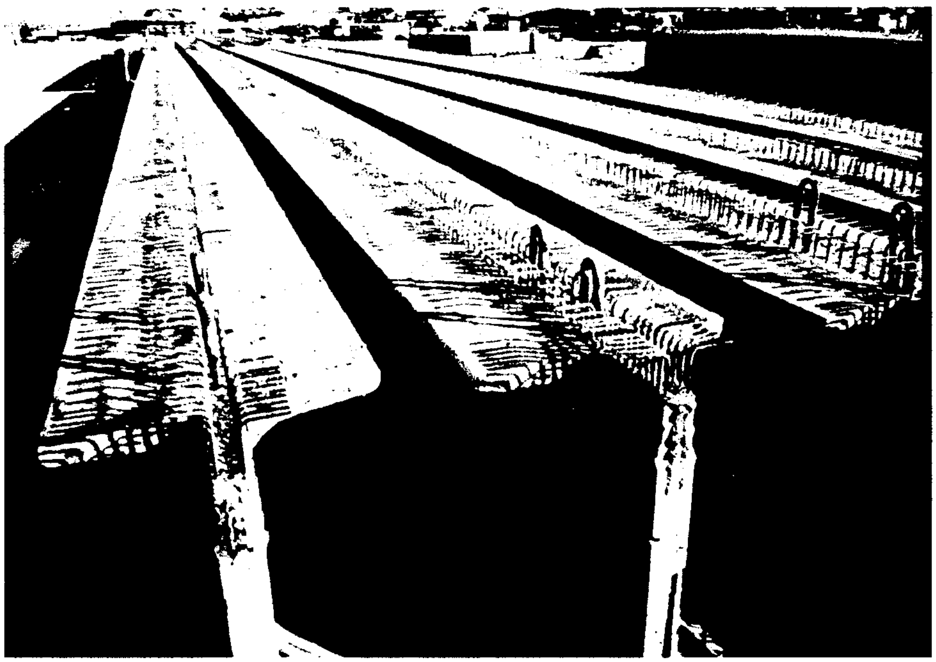


Figure 3.1 Conventional Roughened Interface System

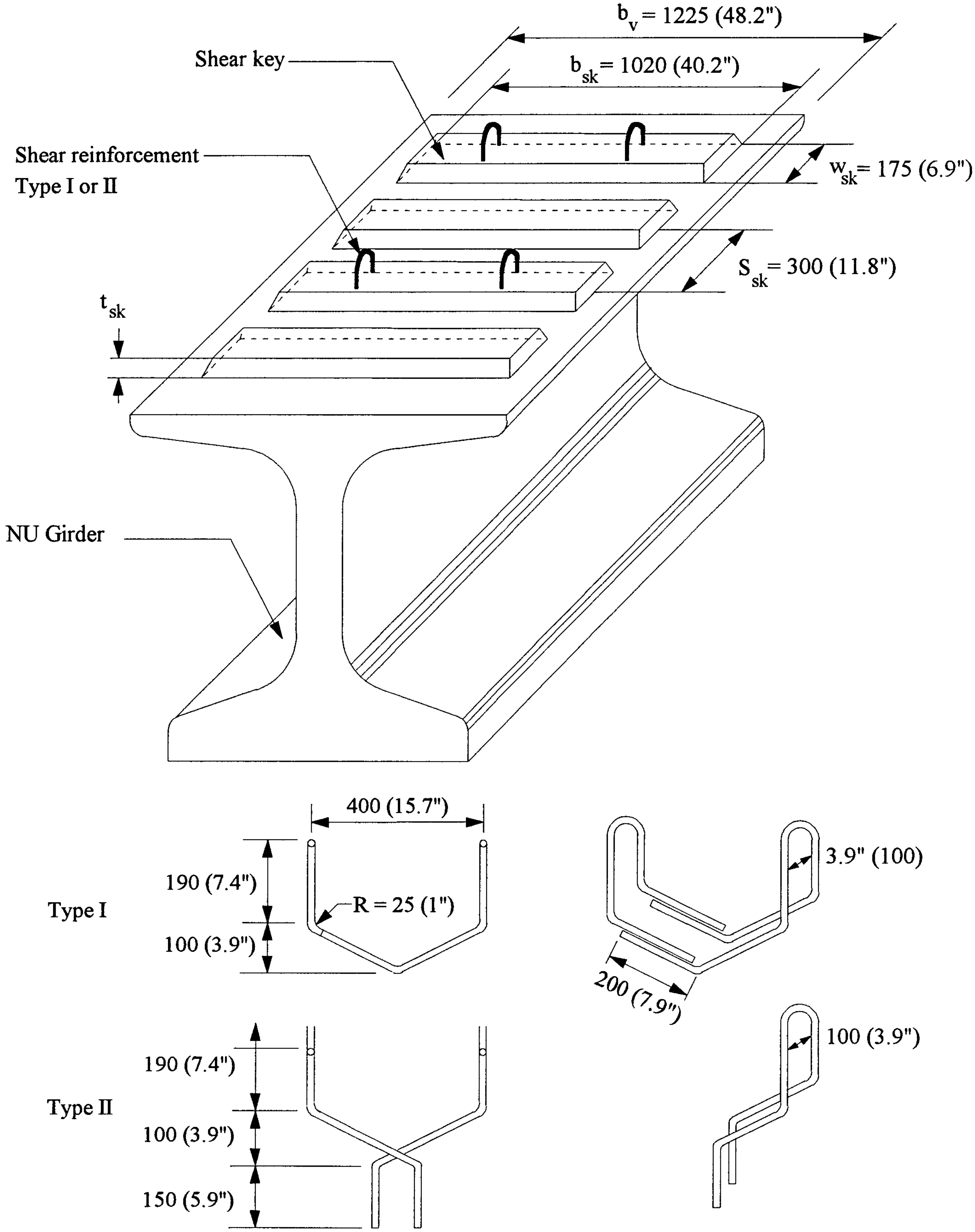


Figure 3.2 Girder Shear Key as Proposed by Kamel (1996)

Figure 3.3 Top View of Shear Key used in the Full Scale Test

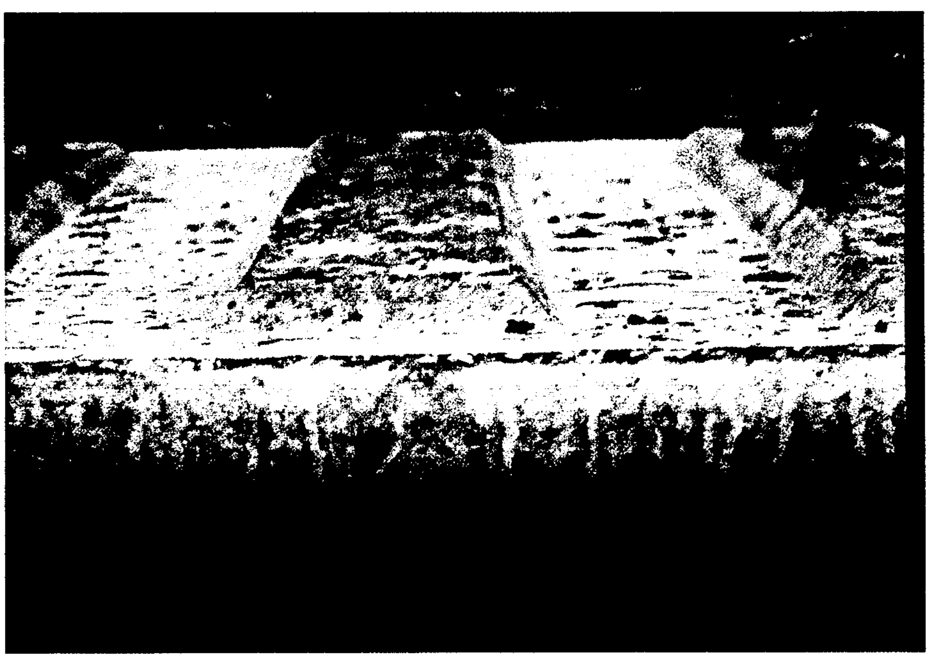


Figure 3.4 Side View of the Shear Key used in the Full Scale Test

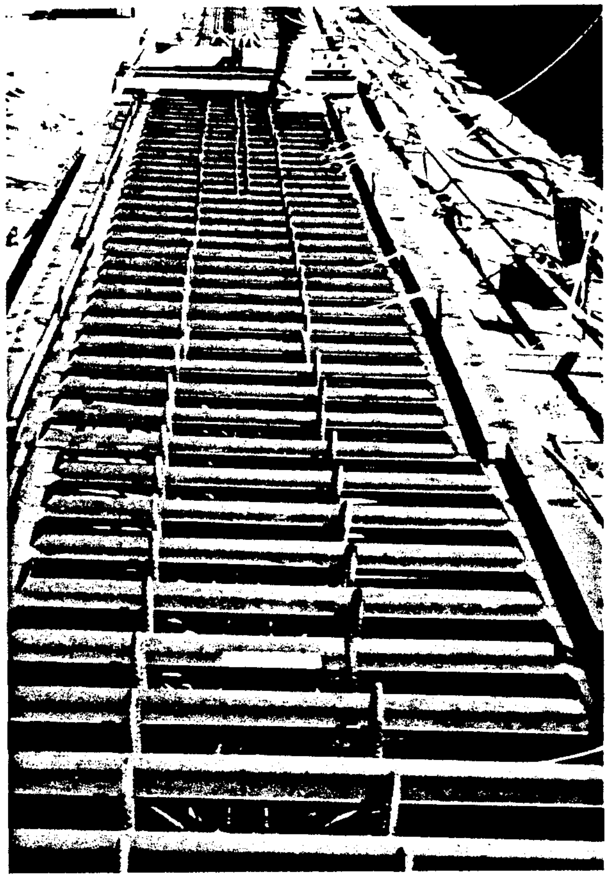


Figure 3.5 Steel Forms used for creating the Shear Keys

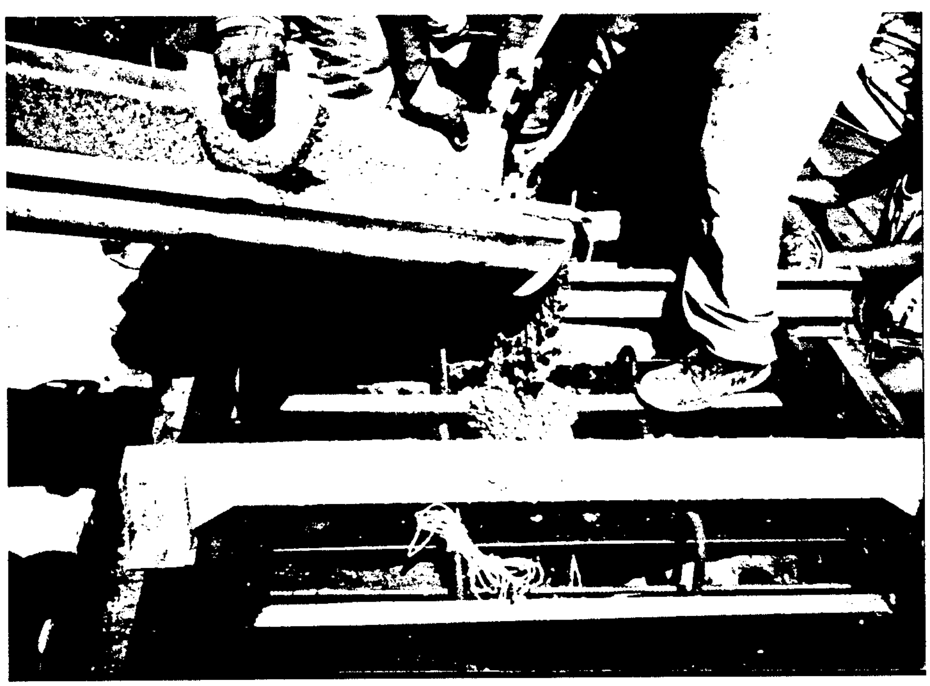


Figure 3.6 Casting Concrete of the Full Scale Test Girder

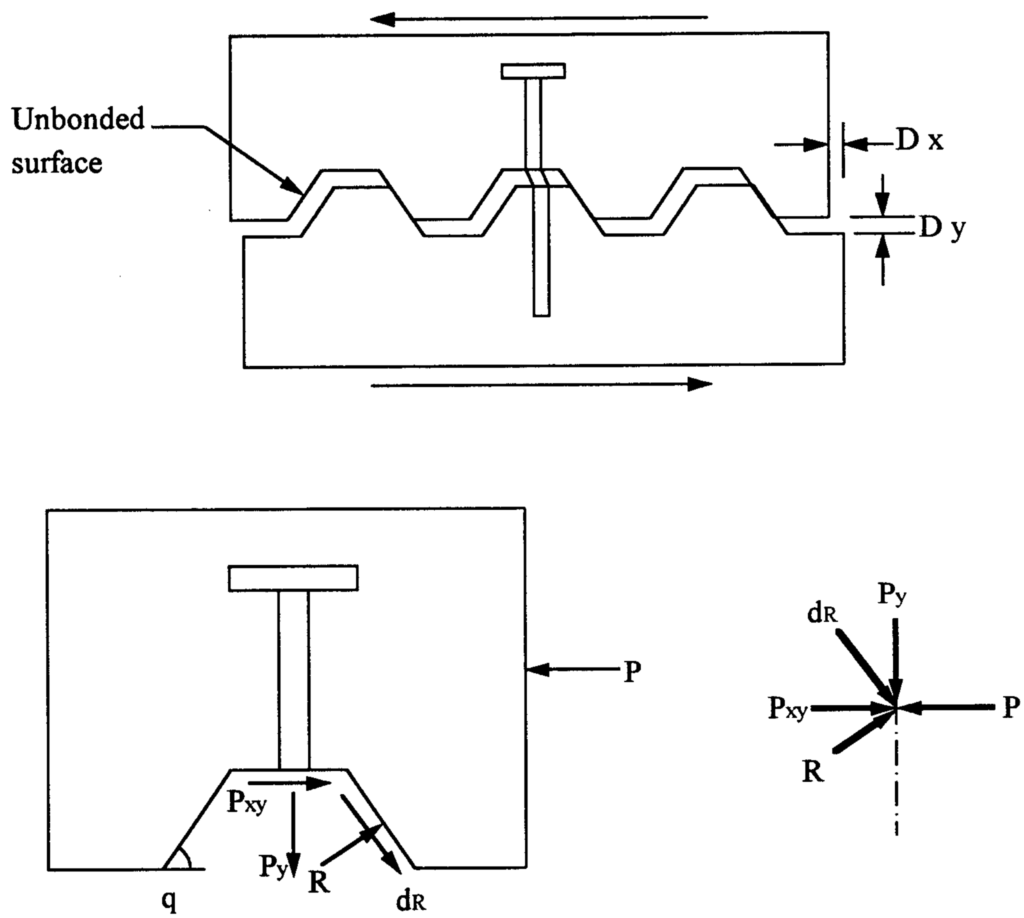


Figure 3.7 Shear Key Mechanism [Kamel (1996)]

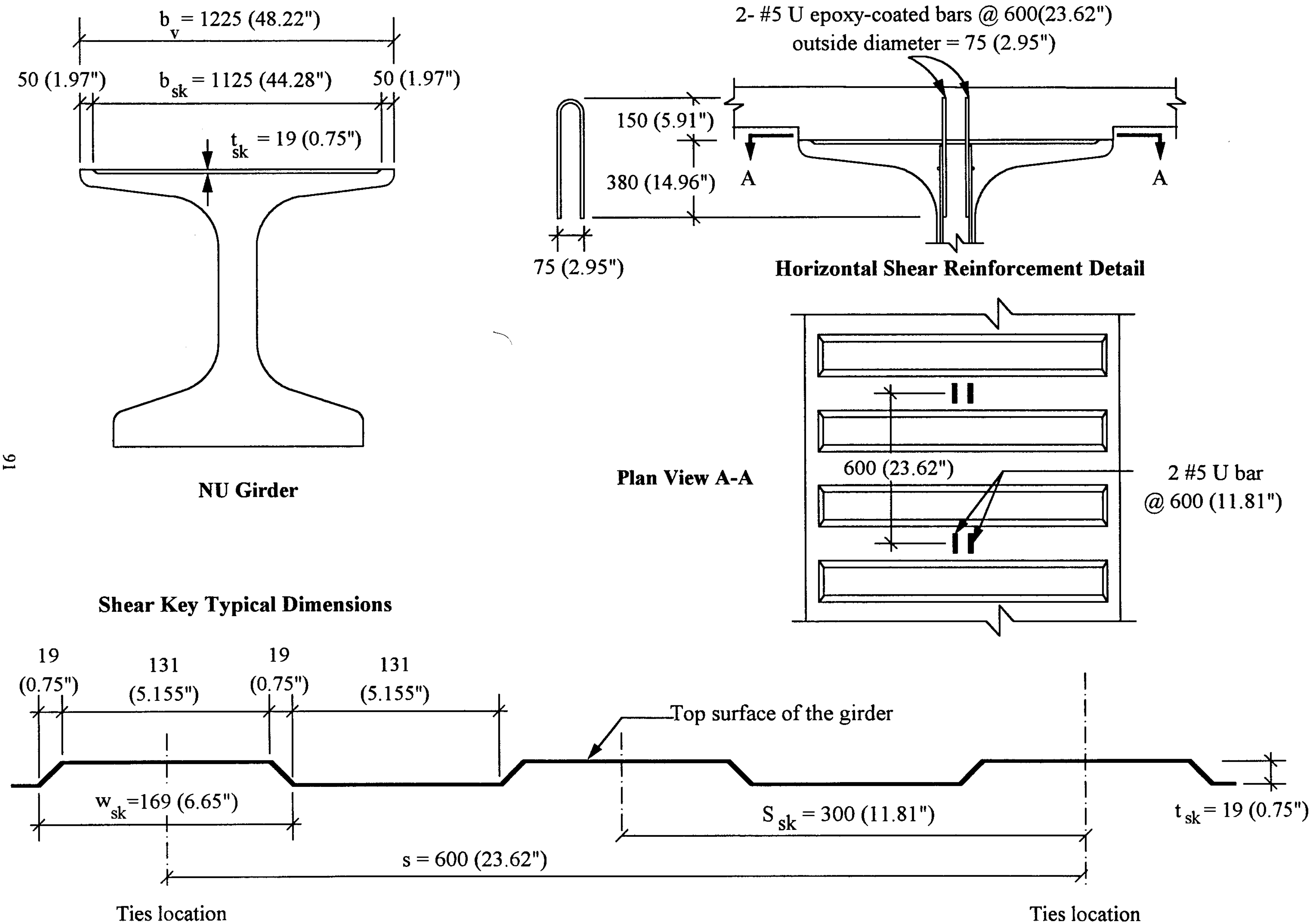
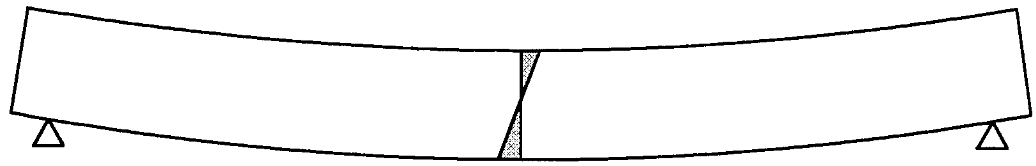
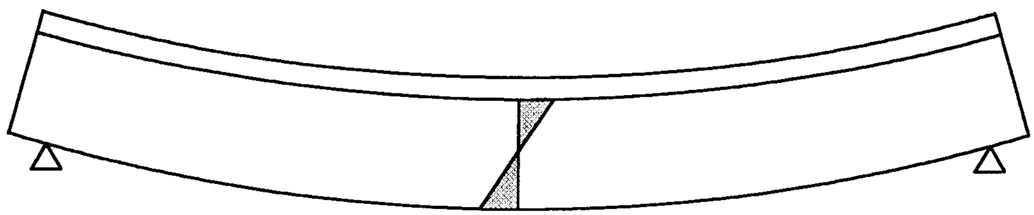


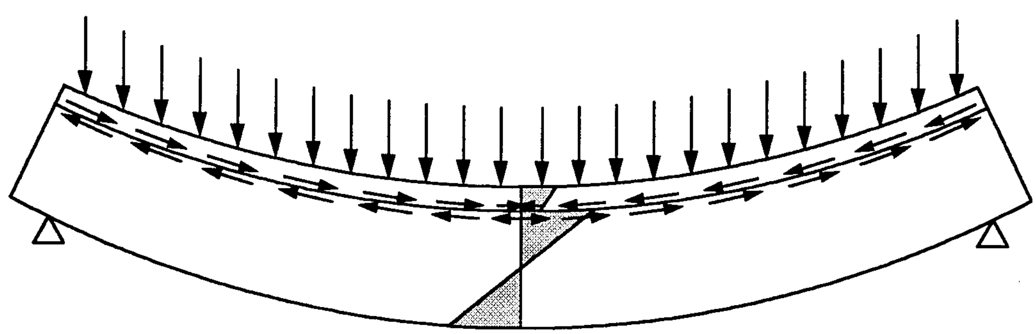
Figure 3.8 Final Details of the Debonded Shear Key System



a) Beam under its self weight



b) Beam under its self weight and slab weight



c) Composite Beam under Superimposed Loads

Figure 3.9 Composite Beam Behavior

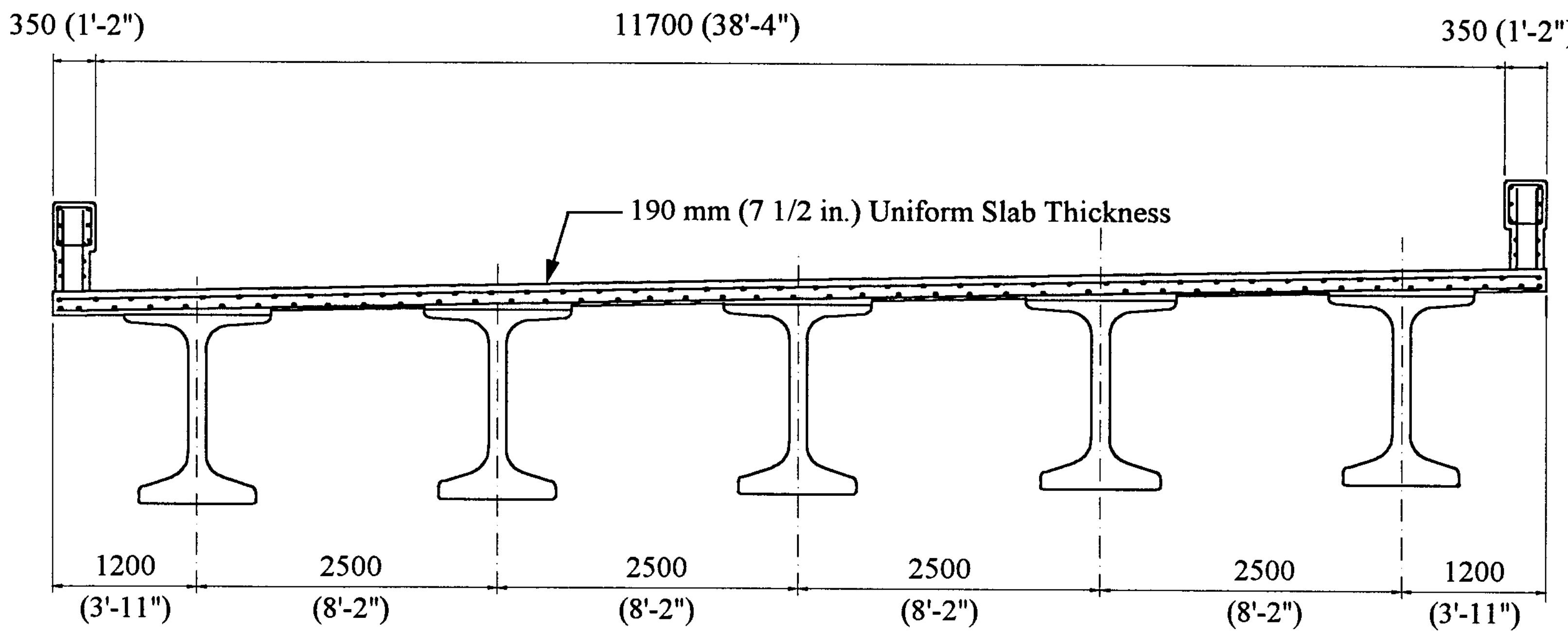
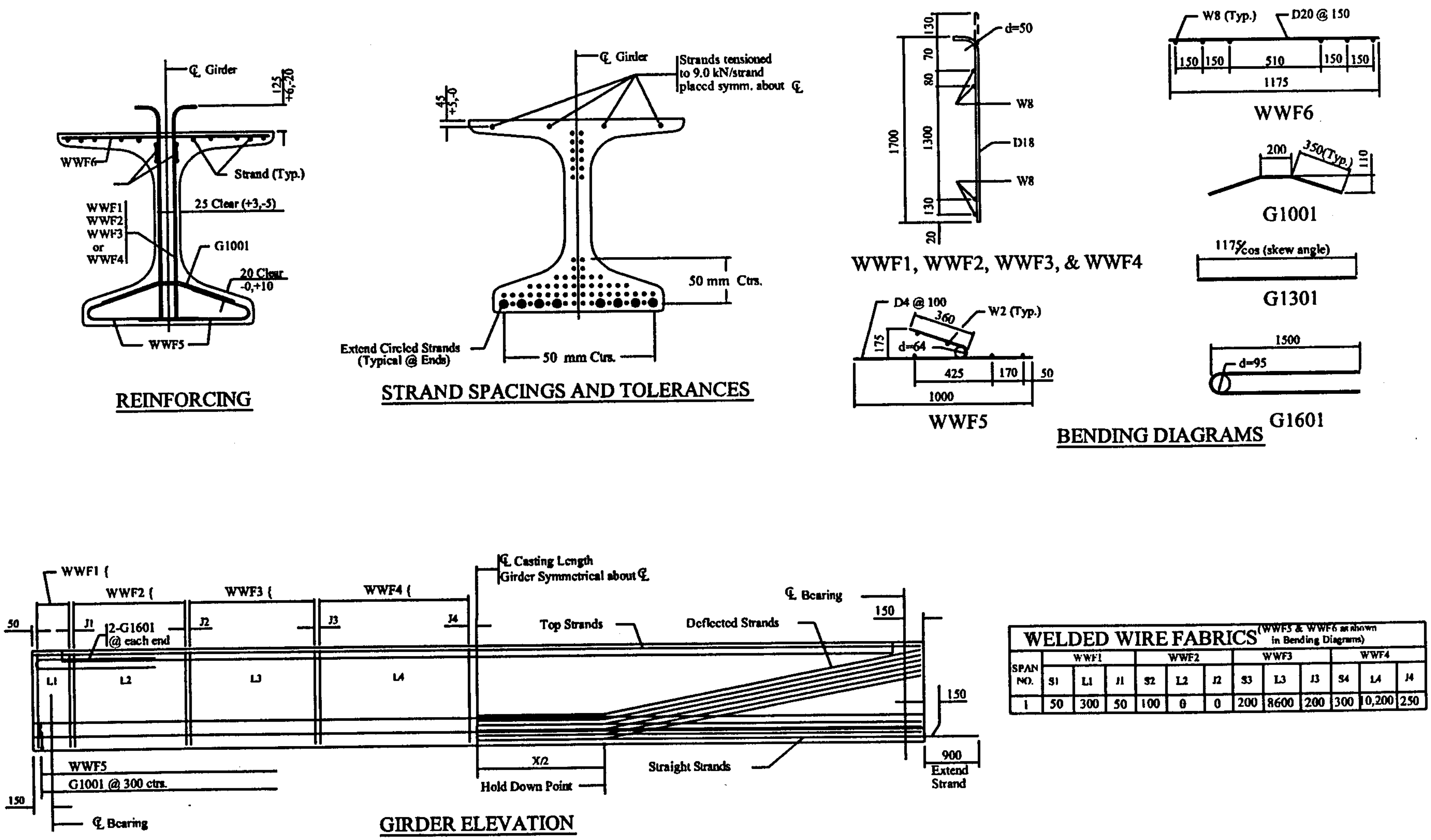


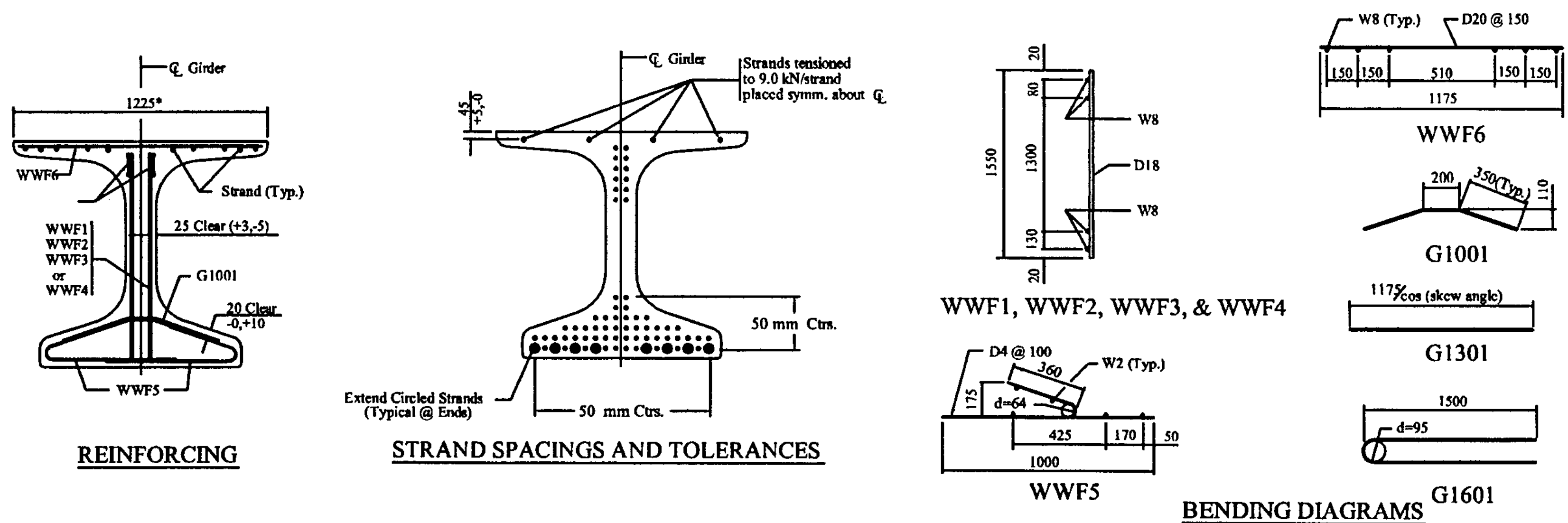
Figure 3.10 Cross Section of the Demonstration Bridge

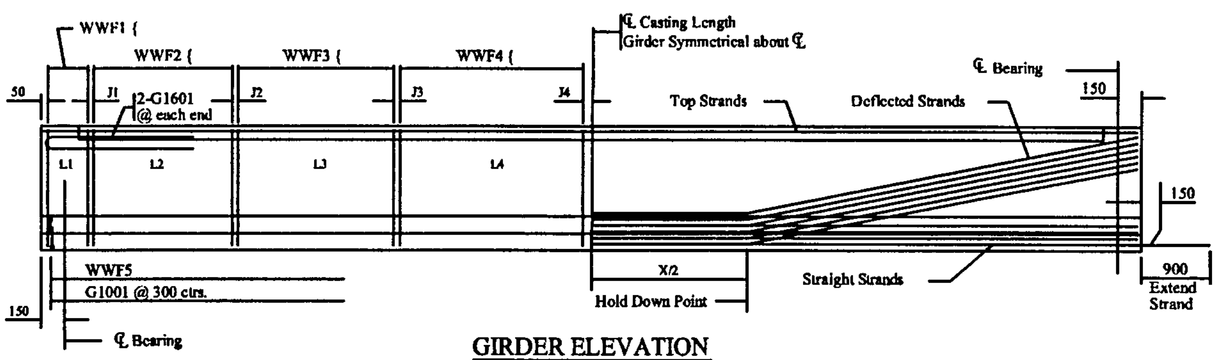


WELDED WIRE FABRICS (WWF5 & WWF6 as shown in Bending Diagrams)

SPAN NO.	WWF1			WWF2			WWF3			WWF4		
	S1	L1	J1	S2	L2	J2	S3	L3	J3	S4	L4	J4
1	50	300	50	100	0	0	200	8600	200	300	10,200	250

Figure 3.11 Conventional Roughened Interface System for the Northbound Structure





WELDED WIRE FABRICS (WWF5 & WWF6 as shown in Bending Diagrams)												
SPAN NO.	WWF1			WWF2			WWF3			WWF4		
	S1	L1	J1	S2	L2	J2	S3	L3	J3	S4	L4	J4
1	50	300	50	100	0	0	200	8600	200	300	10,200	250

Figure 3.12 New Debonded Shear Key Interface System for the Southbound Structure

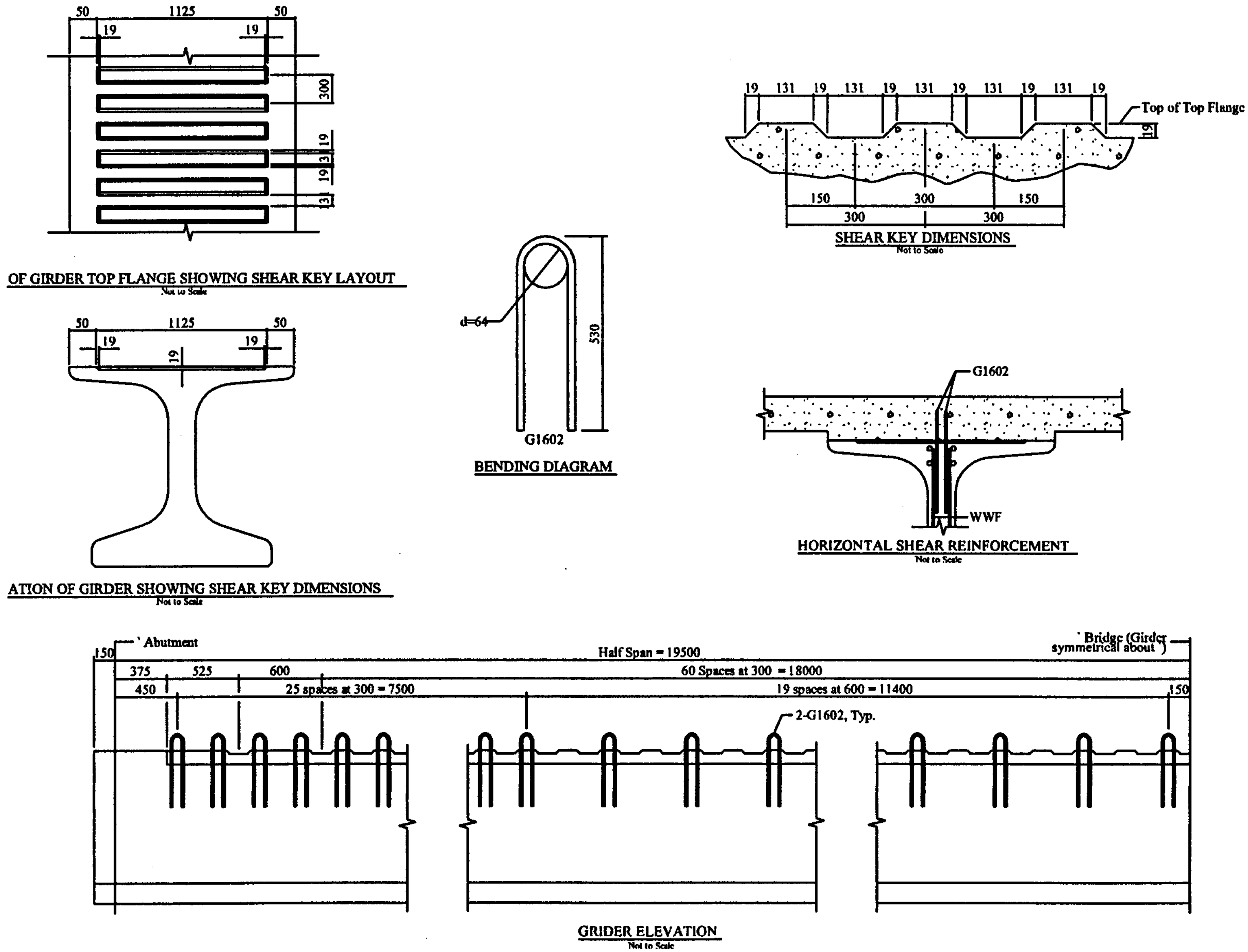


Figure 3.13 Shear Key Details for the Southbound Structure

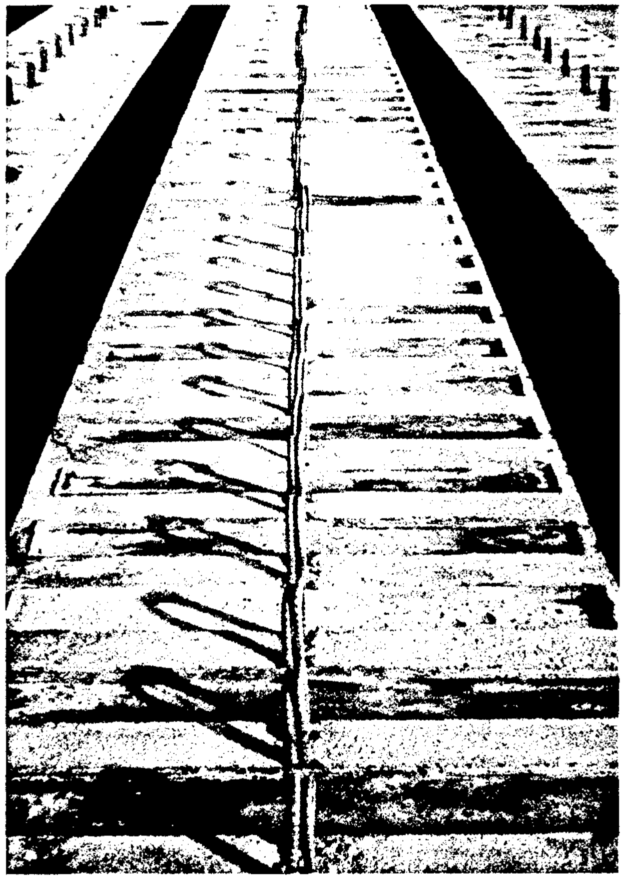


Figure 3.14 Girders with the Shear Keys

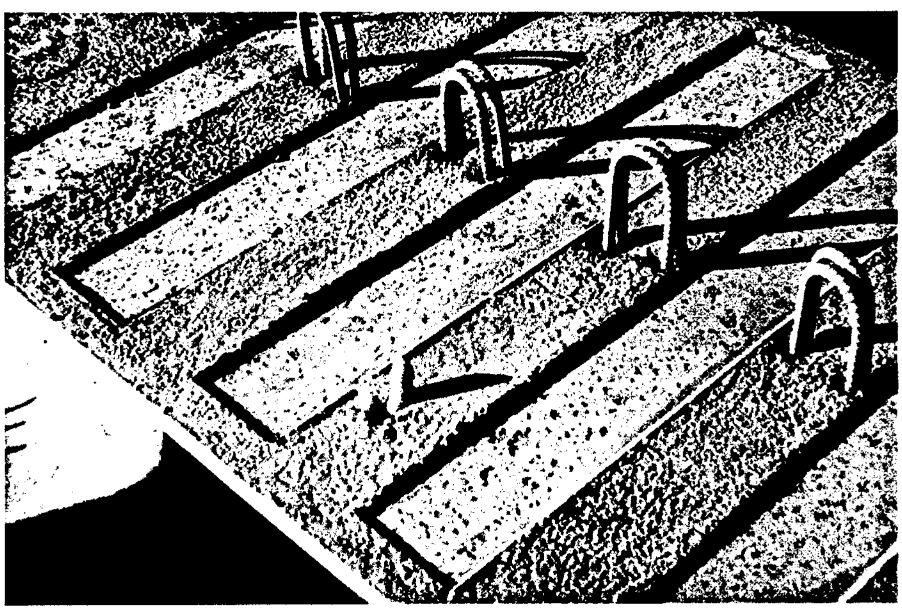


Figure 3.15 Top View of the Debonded Shear Key System

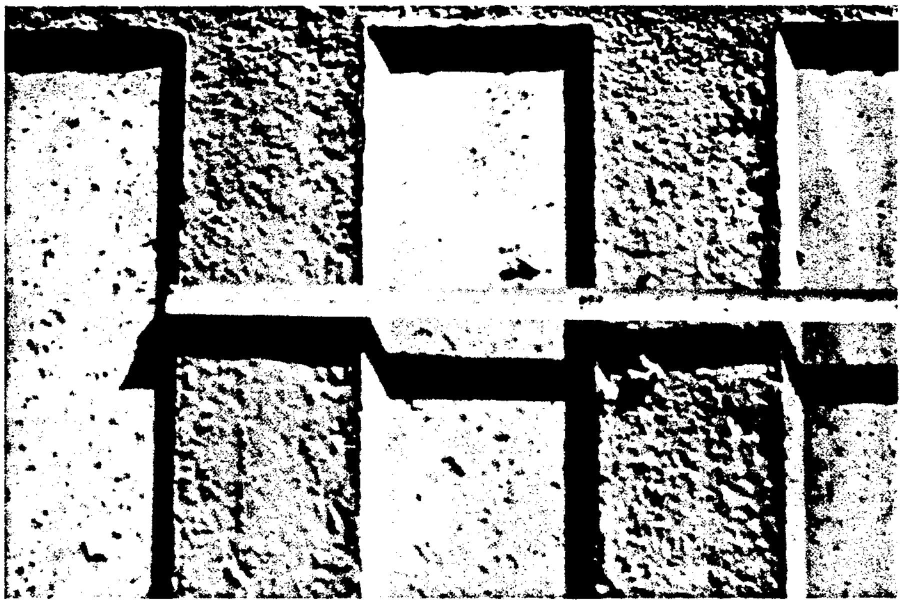


Figure 3.16 Spacing of the Debonded Shear Key System

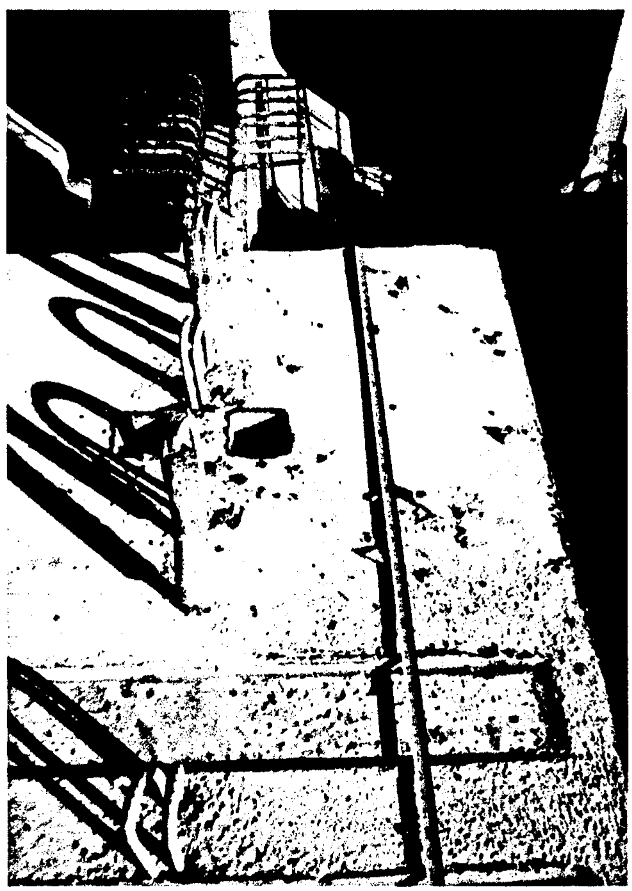


Figure 3.17 Top View of the Girder End

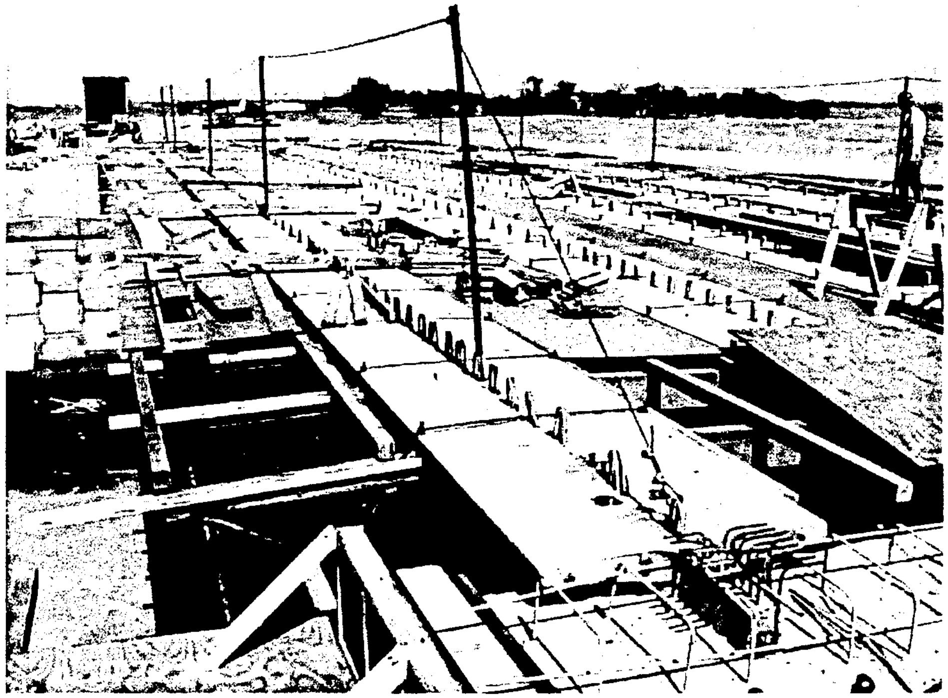


Figure 3.18 Top View of the Southbound Structure

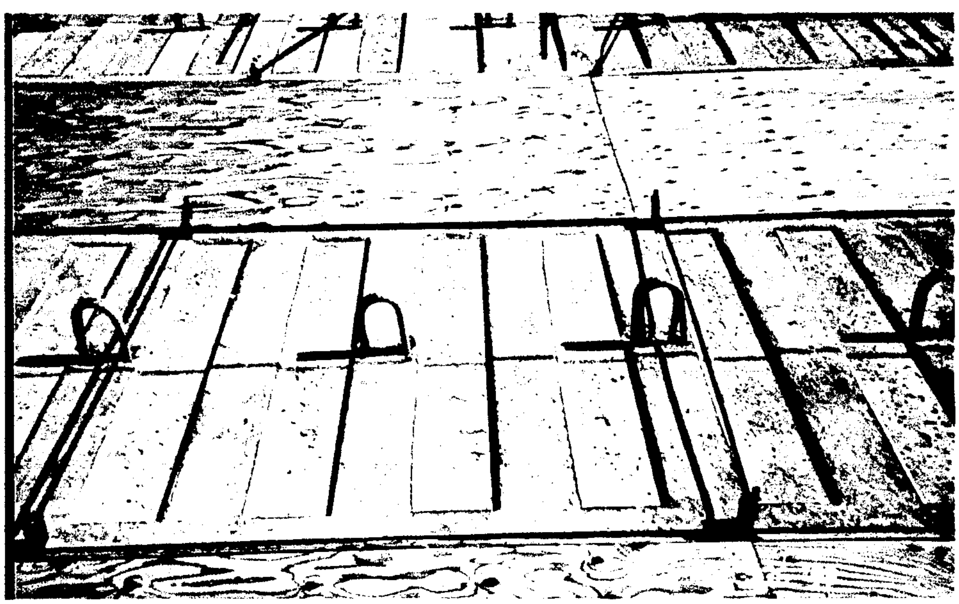


Figure 3.19 Top View of the Debonded Shear Key System

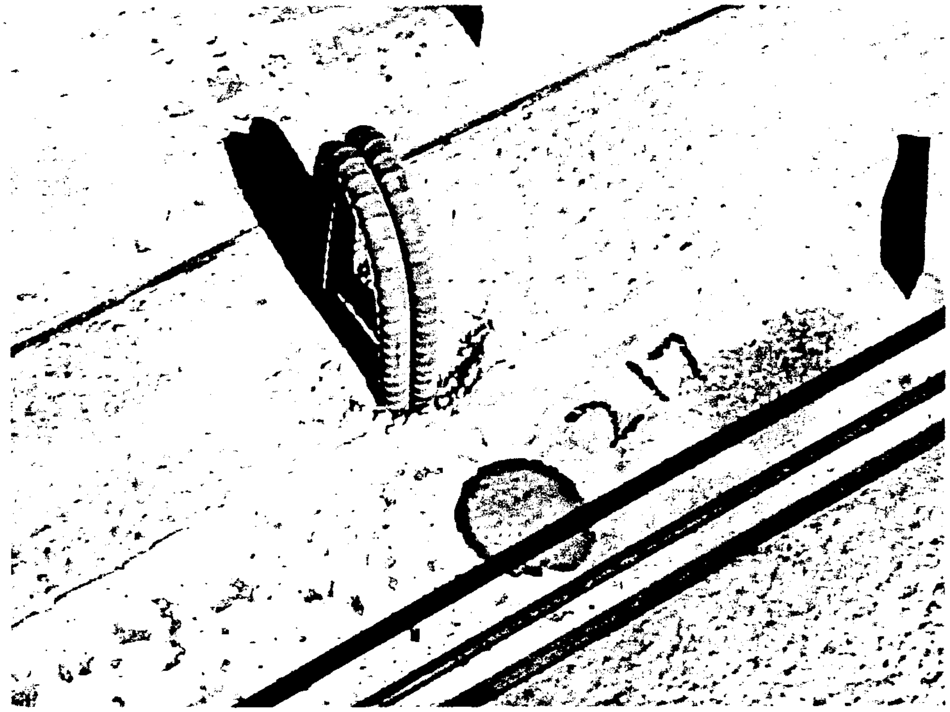


Figure 3.20 U-Shape Shear Connector

Figure 3.21 Top View of the Southbound Structure with the Deck Reinforcement

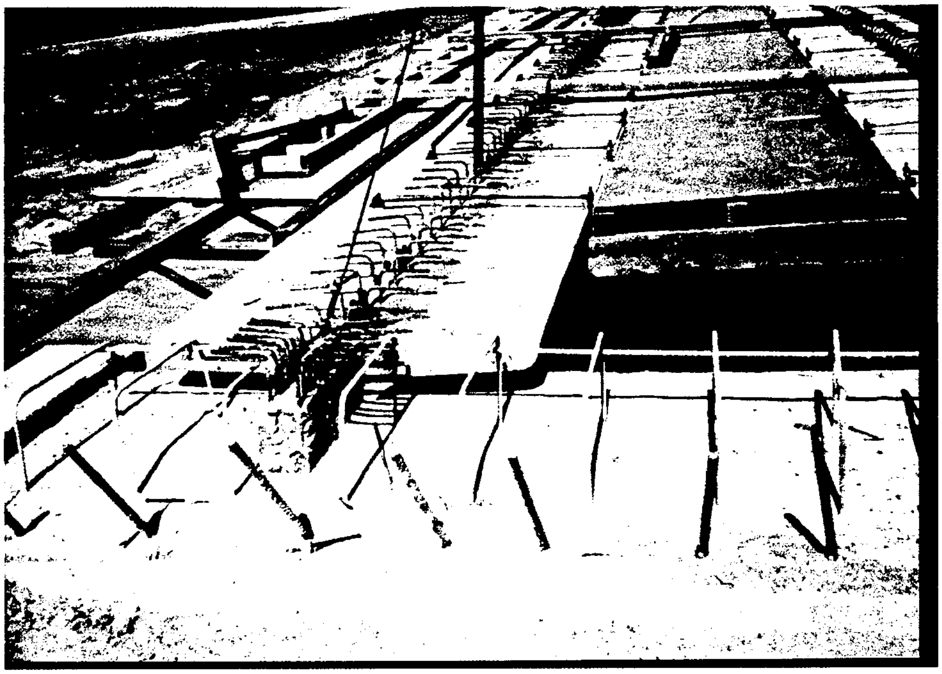


Figure 3.22 Top View of the Northbound Structure

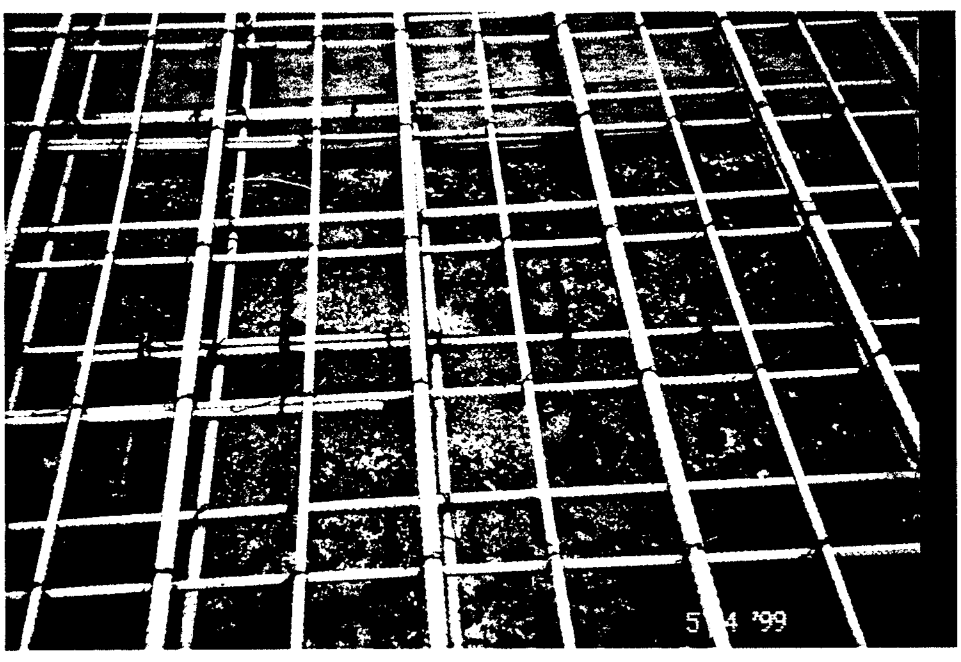
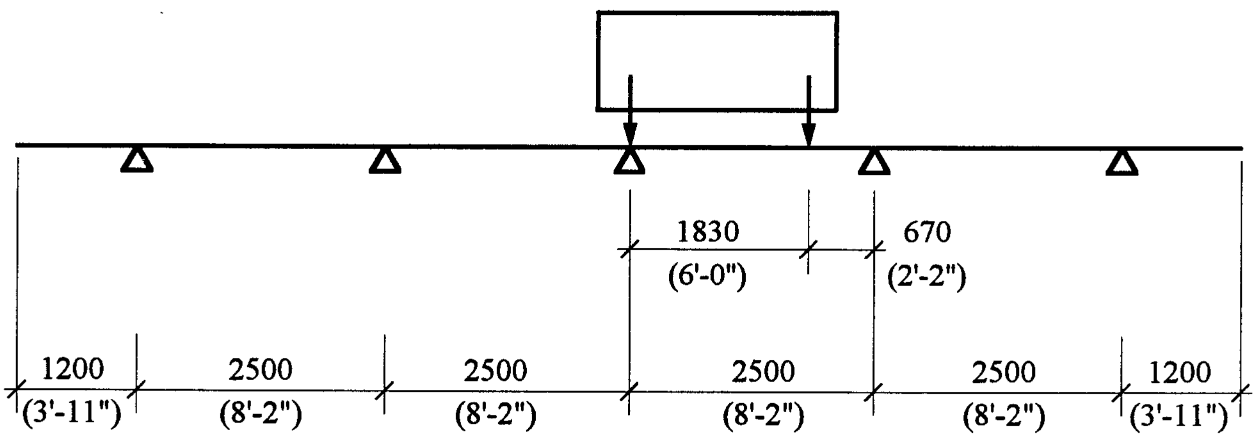
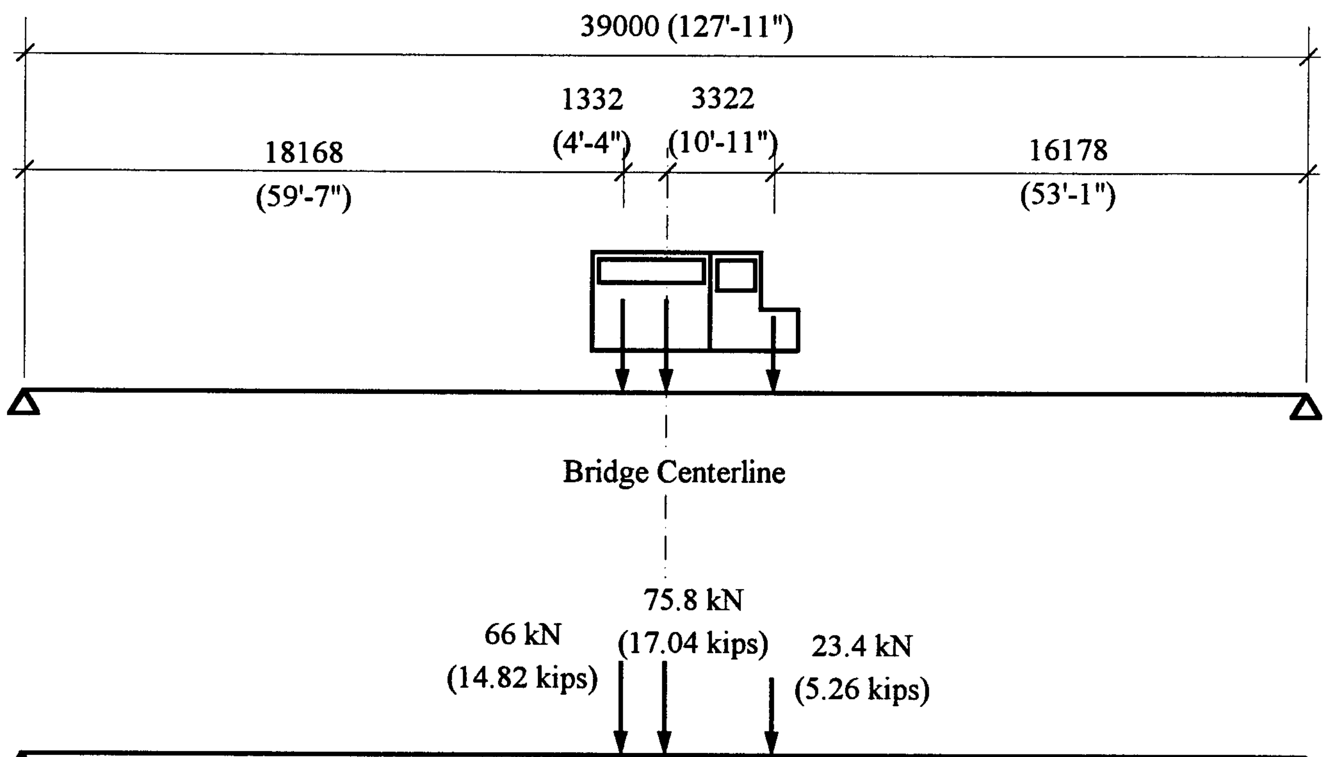


Figure 3.23 Top View of the Northbound Structure with the Deck Reinforcement



a) Transverse Position of the Truck



b) Longitudinal Position of the Truck

Figure 3.24 Field Deflection Measurements

CHAPTER 4

CONCLUSIONS AND RECOMMENDATIONS

4.1 CONCLUSIONS

Two new connection systems, which were developed under NCHRP Project 12-41 [Tadros & Baishya (1998)], have been successfully implemented on bridges in Nebraska. One system is for steel girder/concrete deck connection and the other is for concrete girder/concrete deck connection.

For steel girder/concrete deck connection, a new large 31.8 mm (1¼ in.) diameter shear stud, as shown in **Figure 4.1**, was used to replace the popular 19.1 mm (¾ in.) and 22.2 mm (7/8 in.) shear studs. The new 31.8 mm (1¼ in.) stud provided approximately twice the capacity of 22.2 mm (7/8 in.) stud, which allows use of fewer studs and positioning of the studs in a single row over the girder web.

For concrete girders, a debonded shear key system was used to replace the conventional roughened interface system, as shown in **Figure 4.2**. Shear friction theory was adopted to develop a design procedure for this new connection. Results obtained from extensive laboratory testing [Tadros & Baishya (1998)] and field demonstration of this research project have proved that the new system is a competitive replacement for the conventional roughened interface system.

Field implementation of the new systems on bridges has shown that their performance is comparable with the conventional systems. Using these systems has numerous short- and long-term advantages. Short-term advantages are:

(1) Increase of construction speed of steel girders by welding fewer number of 31.8 mm (1¼ in.) studs compared to 22.2 mm (7/8 in.) studs.

(2) Improvement of construction worker safety because shear connectors for concrete or steel girders are located over the girder web, which gives the construction workers more space to work.

(3) Optimization of concrete NU Girder for horizontal shear design since the full width of the top flange in engaged in the design.

Long-term advantages are:

(1) Minimization of the damage that may occur to the top girder flange during deck removal especially for the concrete NU Girders that have a relatively thin flange.

(2) Enhanced deck removal economy by reducing time and labor. With the new systems, it is possible to saw cut the deck concrete close to the centerline of a girder.

(3) The shear connectors of the concrete girders are protected from corrosion by epoxy coating.

4.2 RECOMMENDATIONS FOR NDOR

The new connection systems shown in this report are the results of a five-year combined laboratory and field investigation, which have shown that these systems are structurally and economically competitive with current conventional systems. Design criteria, specifications, production techniques, and cost analysis are available to bridge design engineers and contractors. Therefore, it is recommended that NDOR adopt these new systems as the standard systems for future projects. Of course, as design engineers and contractors get more experienced with these new systems and as more data is collected from the field, the design specifications and details of these system may be refined in the future.

4.3 RECOMMENDATIONS FOR FUTURE RESEARCH

It is recommended to investigate the use of alternate headed and headless 31.8 mm (1¼ in.) studs for composite steel girders. Although limited test results [Tadros & Baishya (1998)] showed that replacing 50 percent of the 31.8 mm (1¼ in.) headed studs with headless studs resulted in a reduction of the stud capacity of 17 percent, ease of long-term deck removal and replacing may compensate for the increased number of headless studs used. Further study is needed in this area.

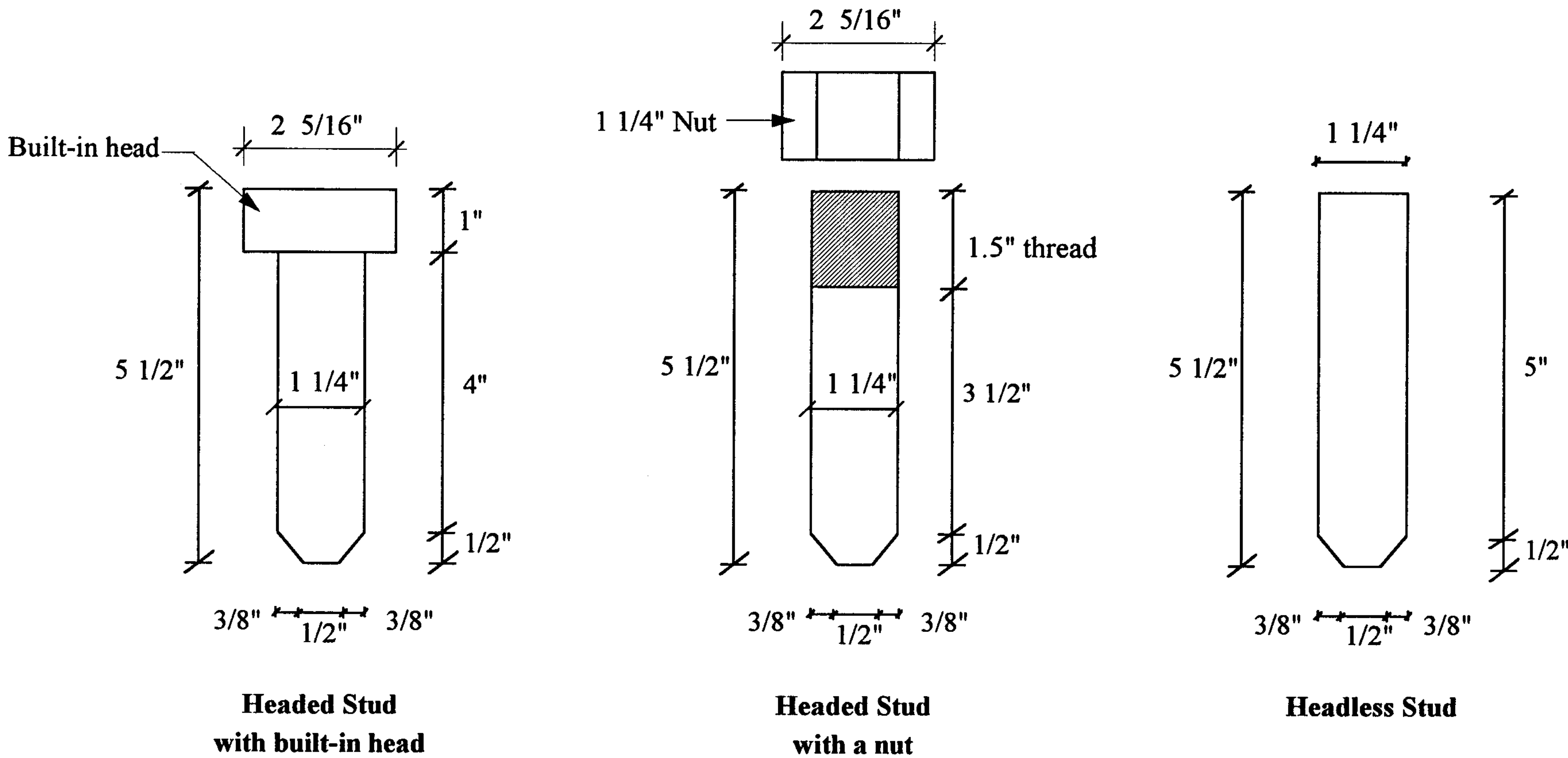


Figure 4.1 Dimensions of the 31.8 mm (1 1/4 in.) Stud

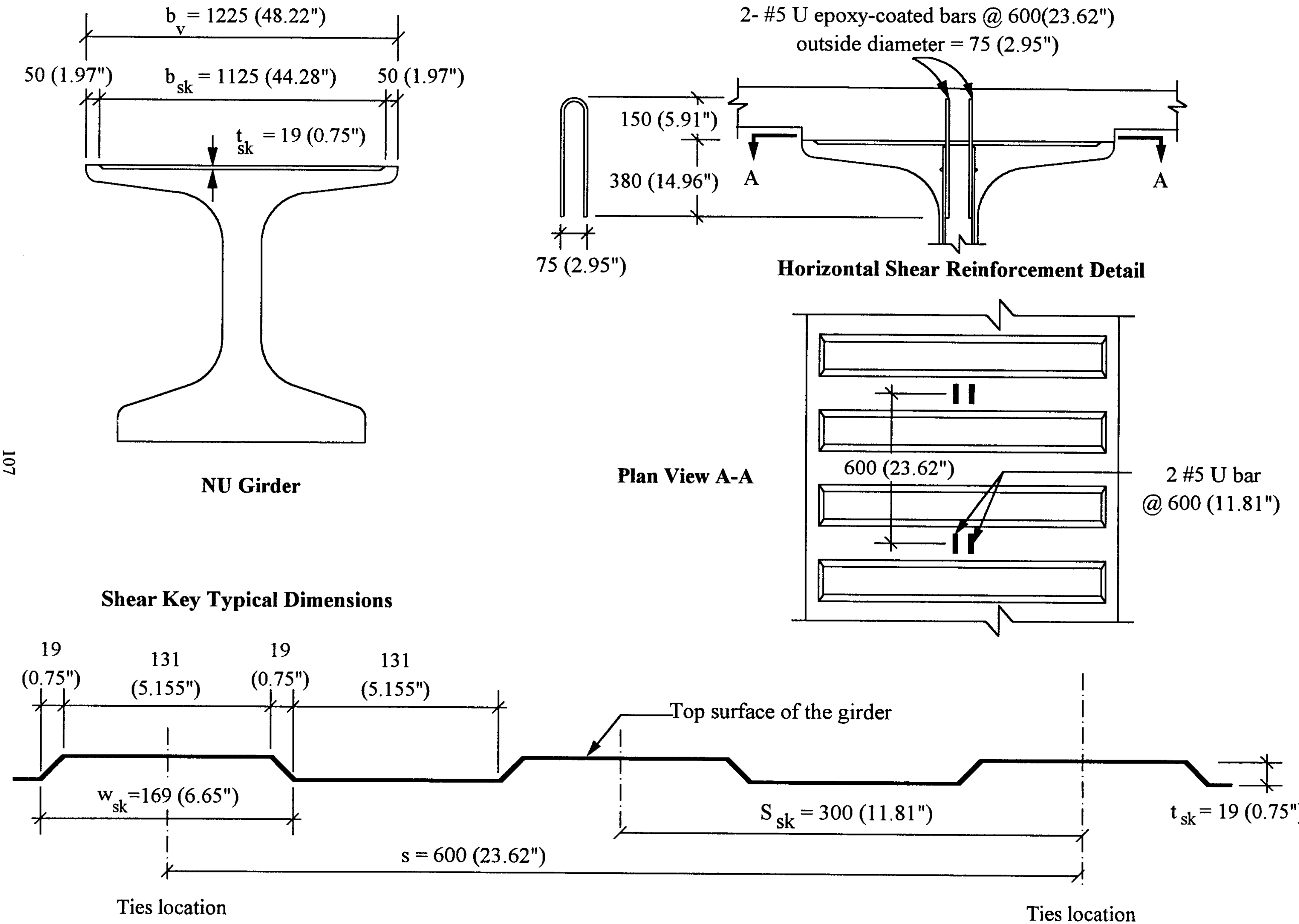


Figure 4.2 Details of the Debonded Shear Key System

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APPENDIX A

LARGE STUD PRODUCERS

The intent of this appendix is to help designers who are interested in using the 31.8 mm (1¼ in.) stud to get information about available producers of this type of large studs. These sources were found after a limited search conducted in this research project. It is strongly recommends that designers should check with local stud producers in addition to the stud producers listed below.

Source #1:

Tri-Sales Associates, Inc.
14901 Chandler Road
Omaha, Nebraska 68138
USA
Phone: 1-800-228-2948, 402-895-5212
Fax: 402/895-3297
Contact person: Tim Rauterkus, Marketing Manager

Source #2:

Continental Studwelding Ltd.
35 Devon Road
Brampton, Ontario L6T 5B6
Canada
Phone: 905-792-3650
Fax: 905-792-3711
Website: www.constud.on.ca
Contact person: Neil Hammill, Marketing Manager

APPENDIX B

COLORED PICTURES OF SELECTED FIGURES

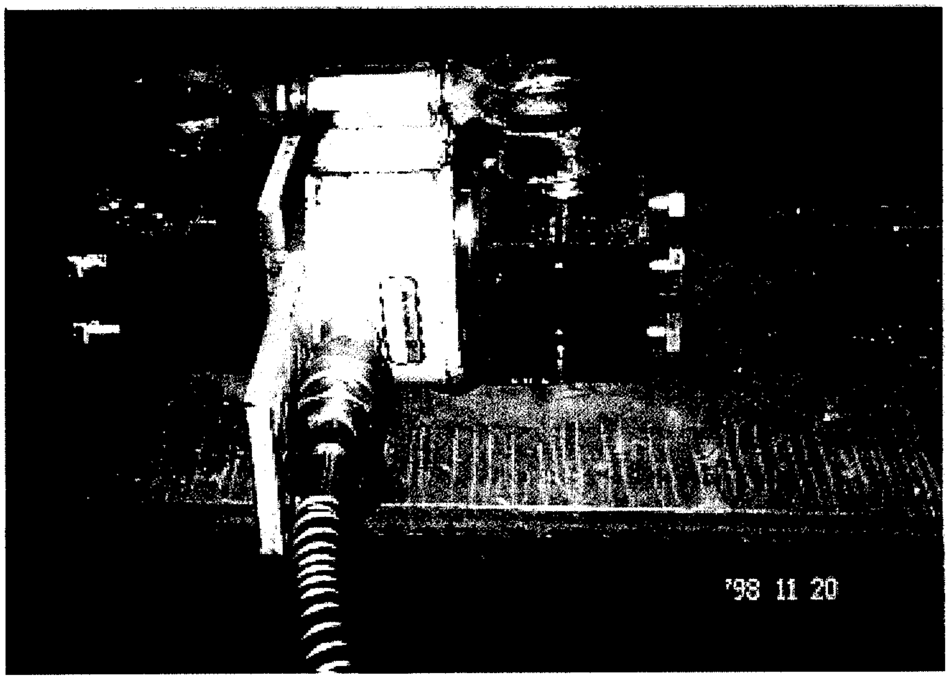
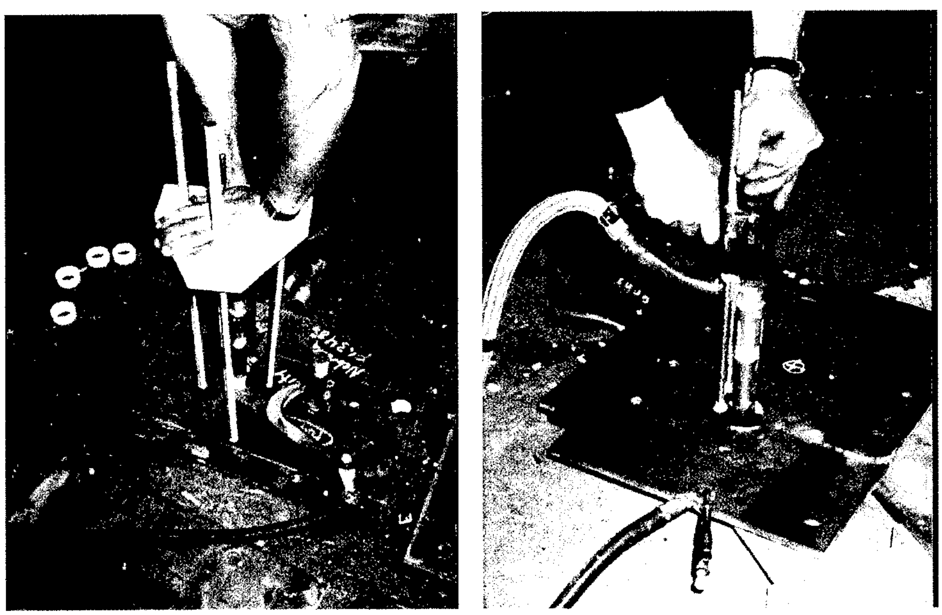
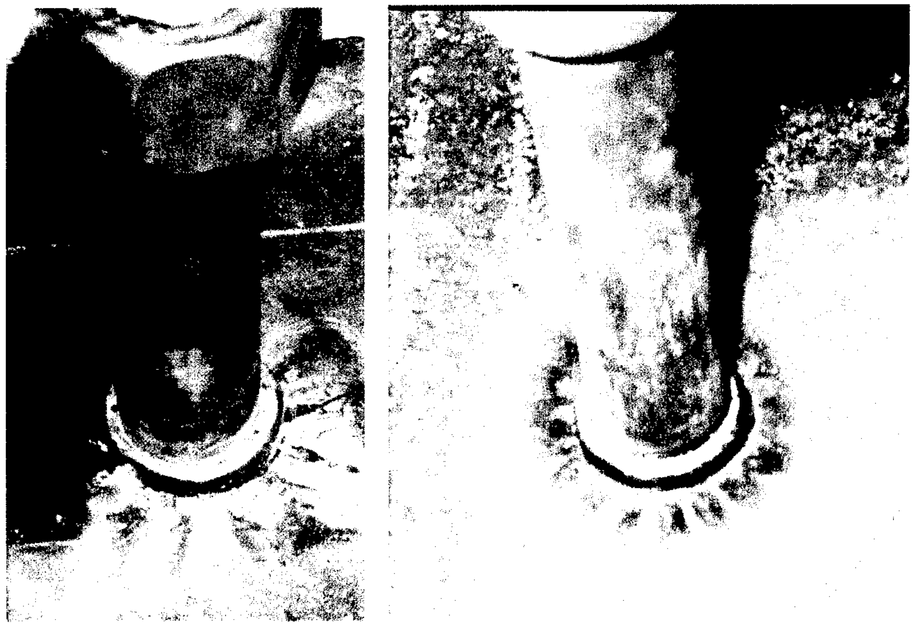


Figure 2.4 Quality Control Setup



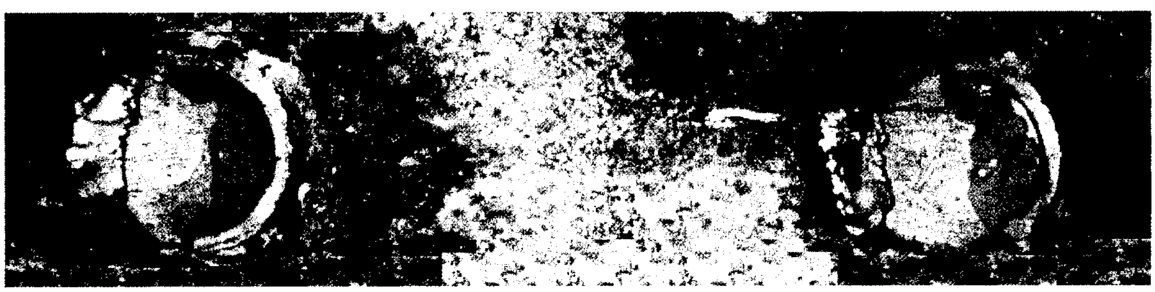
a) With the Tri-Leg Support b) Without the Tri-Leg Support

Figure 2.6 Welding of the 31.8 mm (1¼ in.) Stud

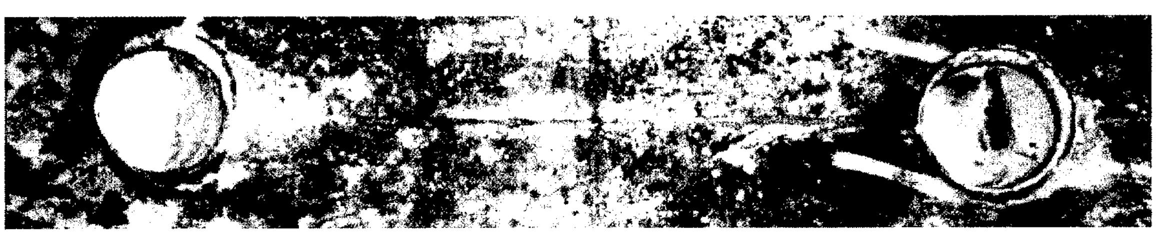


a) 31.8 mm (1¼ in.) Stud b) 22.2 mm (7/8 in.) Stud

Figure 2.7 Stud Welding Quality



a) 31.8 mm (1¼ in.) Stud



b) 22.2 mm (7/8 in.) Stud

Figure 2.13 Fatigue Failure of the Studs

Figure 2.23 Quality Test by Bending the 31.8 mm (1¼ in.) Studs to 45 degrees

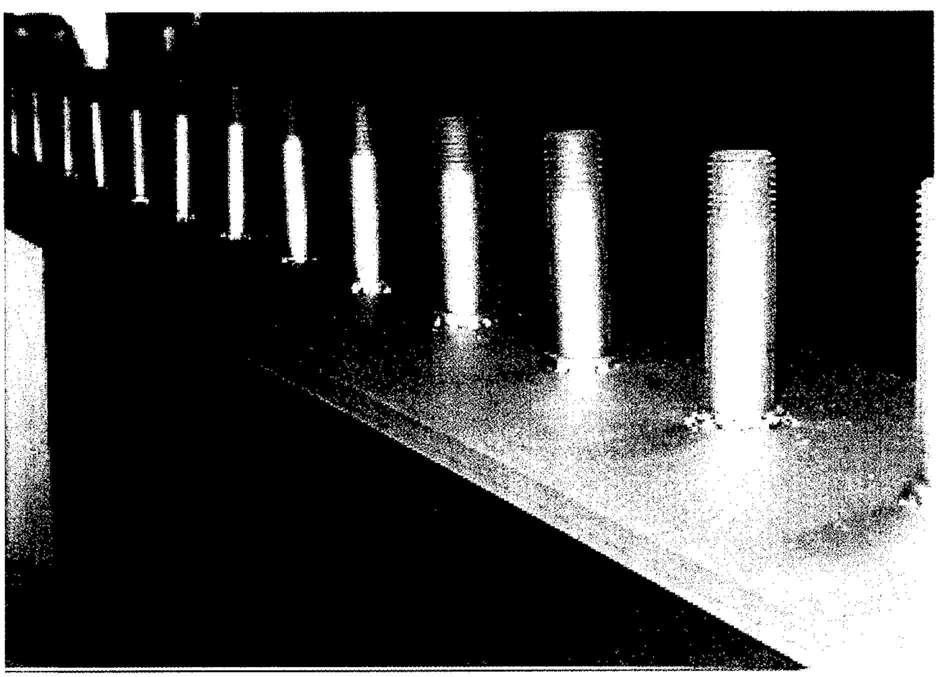


Figure 2.24 Large Studs after Blasting

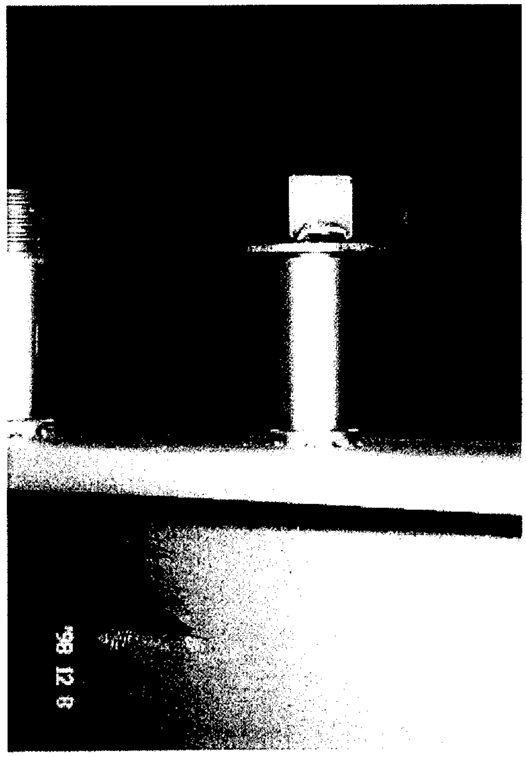


Figure 2.25 Complete Shear Connector

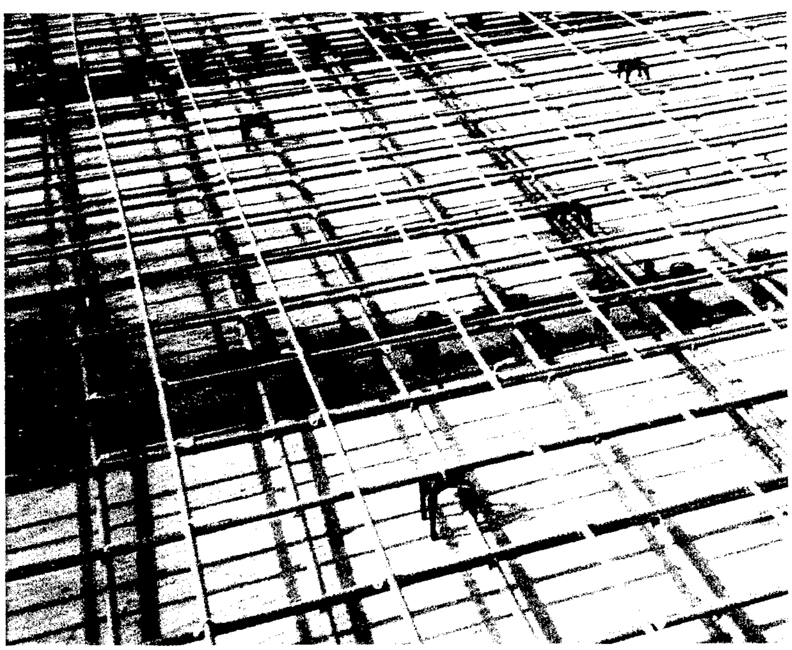


Figure 2.26 Steel Girders with the 31.8 mm (1¼ in.) Studs in Place on the South Span

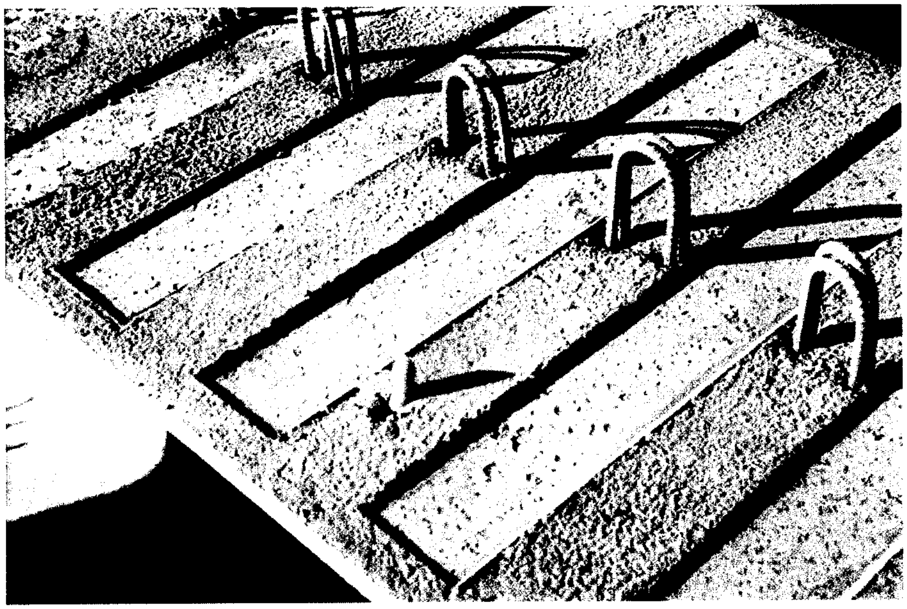


Figure 3.15 Top View of the Debonded Shear Key System

Figure 3.17 Top View of the Girder End

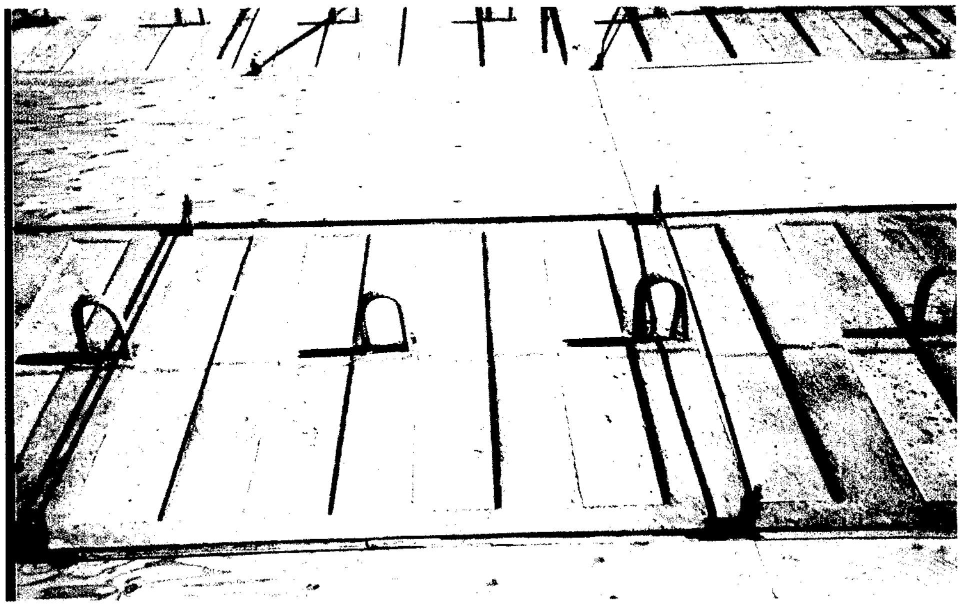


Figure 3.19 Top View of the Debonded Shear Key System

Figure 3.21 Top View of the Southbound Structure with the Deck Reinforcement