BEHAVIOR OF STEEL-TO-CONCRETE CONNECTIONS FOR USE IN REPAIR AND REHABILITATION OF REINFORCED CONCRETE STRUCTURES

APPROVED:

Michael E. Kreger

James O. Jirsa

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by

Julio Jiménez-Pacheco, B.S.C.E.

THESIS

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A mis queridos padres:

"La Negra y La Macha"

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Julio Jiménez
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ABSTRACT

Behavior of Steel-to-Concrete Connections for Use in Repair and Rehabilitation of Reinforced Concrete Structures

by

Julio Jiménez-Pacheco

Supervising Professor: Michael E. Kreger

The need to strengthen non-ductile reinforced concrete frames has been widely recognized as an urgent problem in earthquake engineering. Attaching steel bracing systems and/or steel jackets and plates to existing concrete structures are common approaches used to rehabilitate non-ductile reinforced concrete frames. An effective transfer of forces between the existing concrete members and the attached steel plates or sections is essential for the success of these retrofitting techniques.

Thirty six specimens were tested to study the effect of different variables on the behavior of single-anchor steel-to-concrete connections. The variables studied include: clearance between the hole in the steel element and the anchor bolt, bolt preload, type of filler material used in the void between the hole in the steel element and the anchor bolt, type of material acting as an interface agent (non-shrink grout and epoxy), thickness of interface material, surface treatment of the steel element, type of fastening method (standard and spring washers), bolt inclination, and effect of reversed cyclic loads. Test specimen behavior is presented in the form of load-slip relationships, maximum elastic and utimate capacities, and load-slip envelopes. A comparison of the test results with recommended design provisions is presented to provide guidelines for design of steel-to-concrete attachments used in rehabilitation projects.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1
INTRODUCTION

1.1 Statement of Problem

The hazard posed by a large number of existing reinforced concrete structures having inadequate seismic resistance is one of the most urgent problems faced today by the Structural Engineering Profession. After realizing the devastating effect that a major earthquake may have on their structure, owners throughout the world are becoming more willing to rehabilitate structures in order to provide safety to building occupants and to protect their investment. However, specific guidelines and recommendations for designing economic, constructable systems for correcting deficiencies in existing structures are currently not available for the practicing engineer.

An efficient transfer of force between existing members and attached retrofit elements is critical for many of the currently implemented seismic rehabilitation techniques. In particular, the effectiveness of steel bracing and steel jacketing applications (examples are shown in Figs. 2.1 through 2.3) depends largely on the soundness of the connections between existing concrete members and attached steel elements. The interaction between the original structural components and the steel strengthening plates and sections must be well understood to achieve a successful design of the retrofit systems.

1.2 Objective

The main objective of this study is to investigate the influence of several variables on the load-interface slip response of single-anchor connections between existing concrete and a steel element loaded in tension (direct shear through the steel-concrete interface). Tests were conducted in order to identify methods to:

1) Improve resistance prior to significant interface slip,

2) Reduce slip before reaching peak capacity, and

3) Improve connection toughness

Results of tests are used in the development of basic design recommendations and to identify areas where further research is needed.

Epoxy-grouted anchor bolts installed in a concrete block simulating an existing concrete member acted as the primary component of load transfer. Non-shrink grout and structural epoxy were typically used to fill the void between the hole in the steel element and the anchor bolt, or to act as interface agents between the concrete and steel element. In addition, connections with different fastening components such as spring-loaded washers were incorporated into the experimental program.

1.3 Project Background

This project is part of a National Science Foundation-sponsored research program at The University of Texas at Austin, conducted in collaboration with engineers from H.J. Degenkolb Associates and Englekirk and Sabol, Inc., investigating the use of external steel jackets and plates for repair and strengthening of reinforced concrete frames. A separate testing program involving full-scale tests of columns strengthened with a variety of steel jacketing techniques is currently underway at The University of Texas.

CHAPTER 2
BACKGROUND

2.1 Repair and Strengthening of Structures

The seismic vulnerability and need for rehabilitation of some structures in the nation's inventory of reinforced concrete structures has been revealed by recent earthquakes and improved knowledge in earthquake engineering [2,15]. A large number of buildings designed on the basis of previous codes are being strengthened to comply with all or part of today's standards. Strengthening may be thought of as the implementation of preventive measures to change the response of undamaged structures so that they provide for life safety by preventing collapse, and also limit structural and non-structural damage incurred during future moderate earthquakes. Buildings suffering from previous damage may also require repair and/or strengthening. A significant research effort is underway in the U.S. to develop evaluation and design guidelines for repair and rehabilitation of inadequate structures.

The modified structure must satisfy strength, ductility, serviceability, and socio-economic requirements. Essential facilities such as hospitals, transportation systems, and power plants must remain in operation following a severe ground shaking. Continuity of operations is also essential for high-tech industrial facilities to prevent substantial economic losses [38]. High regard should also be given to the building's functional and aesthetic requirements.

Structures requiring seismic retrofit usually lack sufficient lateral strength and/or stiffness and/or ductility. Selection of a strengthening system to effectively bypass the existing "weak links" in the structure is therefore a challenging task. Current retrofitting techniques include addition of wing or infill walls to moment-resisting frames, attachment of steel bracing systems, and member jacketing. Successful implementation of all of these techniques requires an effective force transfer between new and existing elements.

2.2 Retrofit Techniques Employing Steel Sections and Plates

Retrofitting systems consisting of steel elements attached to the concrete structure, such as steel bracing or jacketing, are sometimes preferred over reinforced concrete jacketing or infill walls for various reasons: 1) steel bracing systems are usually erected from the exterior thereby reducing disruption of activities inside the building, 2) the steel elements add a relatively insignificant amount of mass to the original structure, 3) lateral force resistance may be distributed over the structure to reduce concentrations of shear forces and to eliminate the need for foundation strengthening, 4) steel elements may be prefabricated which reduces on-site labor, and 5) steel jackets offer a convenient means for attaching steel braces when both systems are used in conjunction.

2.2.1 Steel bracing. Steel bracing systems attached to existing concrete frames are designed to carry the induced lateral inertia forces. This technique is recommended for multistory frames with inadequacies at several floor levels [26]. The stiffness of the bracing system must be compatible with the existing concrete structure. A large portion of the load should be carried by the strengthening system at all deflection levels to prevent damage of the concrete frame. Because local deficiencies often necessitate modification of some frame members, steel bracing may be used in combination with steel jacketing.

An effective transfer of forces through steel-to-concrete connections is essential for sound hysteretic response of the strengthening scheme. Connections must ensure that capacity is fully developed in the bracing cross members. Collector members are typically attached to the existing structure to transfer forces from floor diaphragms to the steel braces and from the braces to the frame columns at each floor level (Fig. 2.1). Collector members consist of steel plates or rolled shapes fastened to the structure with retrofit anchor bolts.

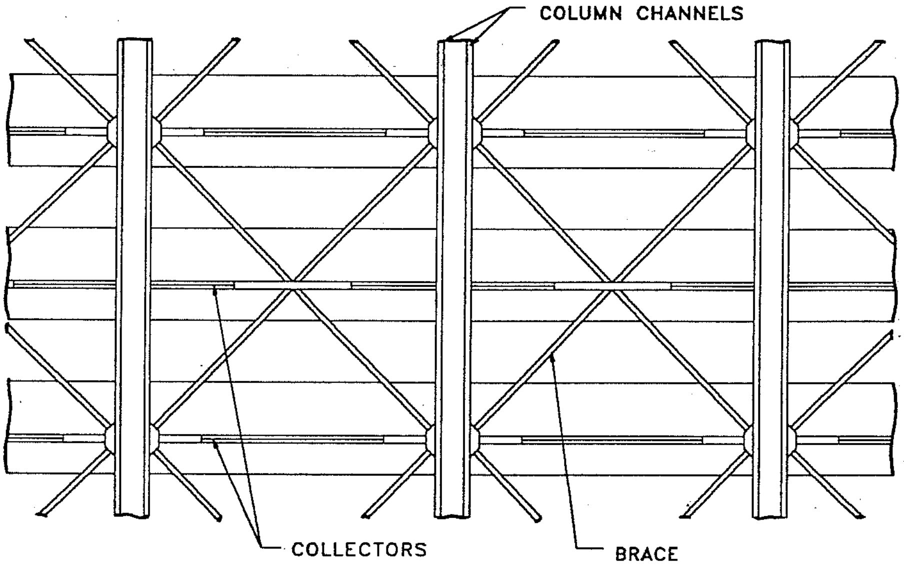
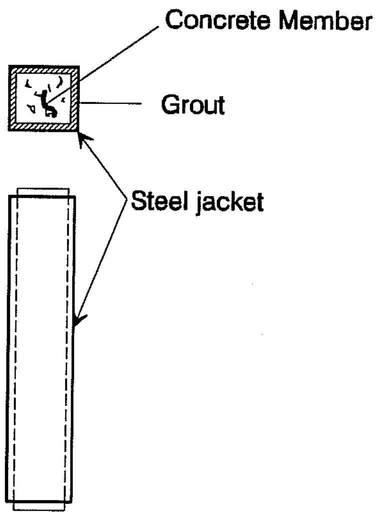


Figure 2.1 Steel bracing retrofit scheme [11]

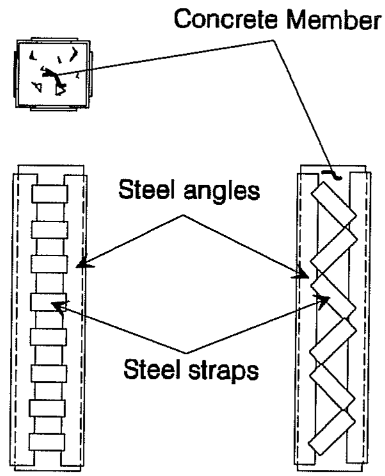
2.2.2 Steel jacketing. The behavior of existing concrete members may be enhanced by attaching steel elements or plates on their exterior [15,22]. Steel jacketing creates a concrete-steel composite section to increase the confinement, shear strength or flexural strength of columns, or to increase the flexural strength of beams by using steel plates as external longitudinal reinforcement. In addition, continuity of reinforcement and improved joint behavior may be provided by connecting steel plates attached to beams to jackets on columns.

The most common approaches for improving strength and/or ductility of existing concrete members include the following (See Fig. 2.2):

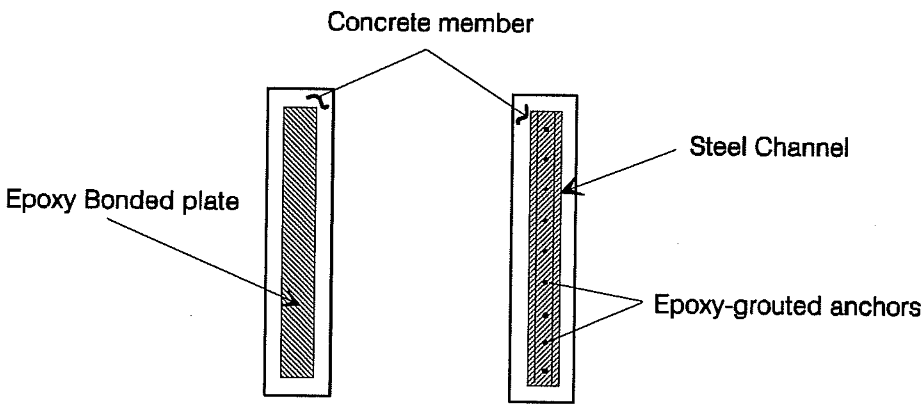
1) Encasement of the member with steel plates or rolled shapes having non-shrink mortar or grout placed between the member and strengthening element,



(a) Encasement with steel plates and expansive grout



(b) Steel jacketing using angles and straps



(c) Strengthening with steel plates or sections attached with anchor bolts and/or structural epoxy

Figure 2.2 Steel jacketing techniques

2) Placement of a skeleton of angles and straps around the member,

3) Attachment of plates to the members with adhesives and/or bolts.

Another important application consists of attaching a steel collar on opposite sides of a joint as indicated in Fig. 2.3. This technique improves joint confinement and capacity, and enhances shear transfer with floor diaphragms.

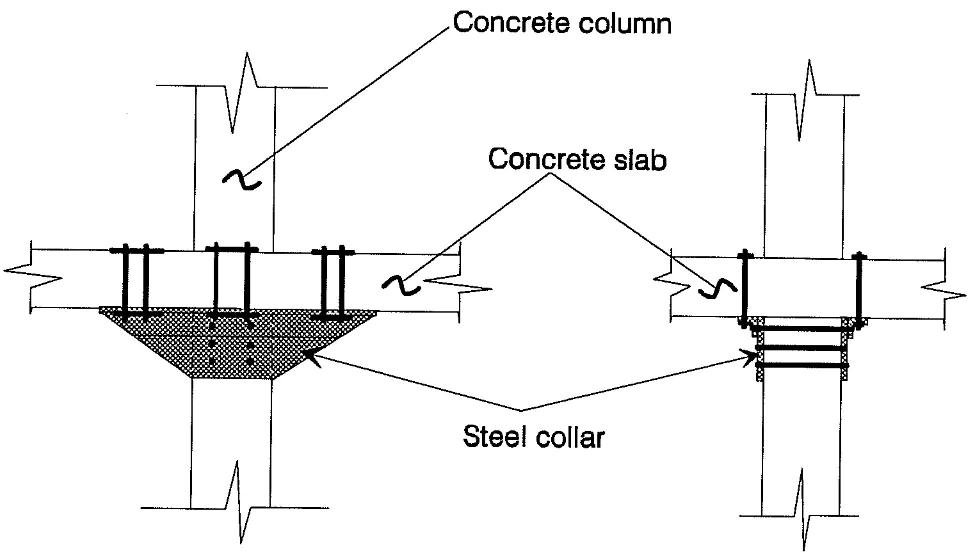


Figure 2.3 Steel collar for moment transfer between column and floor

Steel jacketing has been used extensively in rehabilitation of reinforced concrete structures. However, little data are available on its effectiveness, particularly on the capacity and behavior of the connections used to attach the jackets. Connections should be designed to guarantee composite response. Ductile failure of the modified members should occur before connection capacity is reached.

2.3 Steel-to-Concrete Connections

2.3.1 Retrofit anchors. The most common approach to attach steel elements to existing reinforced concrete members is through the use of retrofit anchors: wedge (expansion or undercut), and epoxy anchors (Fig. 2.4). Another approach to attach steel plates to concrete members is adhesive bonding.

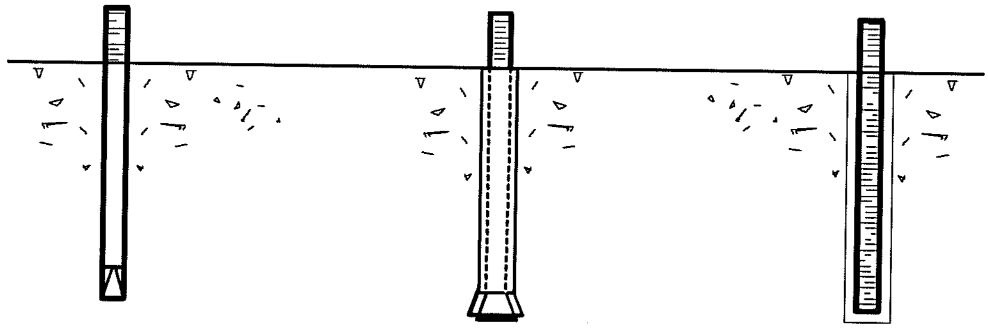


Figure 2.4 Types of retrofit anchors

2.3.1.1 Expansion and undercut anchors. Load in expansion anchors is transferred by friction. When torque is applied to the nut, the bottom sleeve expands against the sides of the hole, creating a friction force between the bolt and the base material. Load in undercut anchors is transferred from the anchor to the concrete by bearing on the undercut portion of the hole [19].

The behavior and capacity of wedge anchors for seismic applications is questionable since manufacturers base their design capacity on static, monotonic tests [40]. Anchors in retrofitted structures are likely to experience combinations of repeated shear and tensile forces during earthquakes. Data from an experimental program conducted at the University of Texas on the behavior of retrofit anchors showed stiffness degradation of undercut and expansion anchors subjected to impact tensile loading due to increased slip. Adhesive anchors exhibited no reduction in secant stiffness up to yield level impact loads [18].

2.3.1.2 Epoxy anchors. Recent investigations have revealed good performance of epoxy anchors for retrofit applications [8,17,22,26,32,45]. An epoxy anchor consists of a threaded steel bolt bonded in place in a drilled hole in existing concrete by using a two-component structural adhesive. Load is transferred from the anchor through the adhesive to the concrete along the entire embedment length. Since bond between the adhesive and concrete is essential for load transfer, proper hole cleaning is of foremost importance [18,21,32,45].

2.3.1.3 Adhesive bonding.

Advantages. The main advantages of bonding steel elements to existing concrete are: 1) bonded connections reduce stress concentrations by distributing the transfer of shear forces throughout the connection, 2) bonded connections have shown high elastic capacities [45], and 3) the need for time-consuming close-tolerance machining might be reduced. Experimental results of bonded plates providing bottom reinforcement to concrete beams have shown satisfactory results for monotonically applied loads [27,41,43]. However, the response of such attachments to severe cyclic loading has not been demonstrated. Adhesives used in combination with fasteners to distribute loads or "seal" voids might provide highly reliable joints. Research at The University of Texas has shown superior behavior of bolted steel-to-concrete connections incorporating adhesives over connections using mechanical connectors alone [45].

Epoxy grouts. Epoxy adhesives are made by mixing an epoxy resin and a curing agent. Proper application and thorough mixing of the two components are very important. Epoxy resins are thermosetting plastics requiring heat and exhibiting slight shrinkage while curing. The resulting adhesives are subject to creep which increases with temperature and humidity [1,28,31]. Creep is generally not considered detrimental in seismic retrofit applications since the material will be utilized only on a transient basis at the time of ground shaking. However, creep of the epoxy in adhesive anchors may result in the loss of bolt preload [21].

Special surface preparation of both contact surfaces is essential. The most commonly recommended method of surface preparation for both steel and concrete is sandblasting [4]. It is advisable to bond the cleaned surfaces immediately or within eight hours of substrate preparation [4].

2.3.2 Behavior of ductile connections in shear. As discussed above, the success of strengthening programs using steel jackets and plates depends largely on the performance of the connections between structural steel and reinforced concrete. Steel-to-concrete connections should provide high stiffness at design loads, and adequate strength and ductility at ultimate loads.

Ductility is defined as the ability of a structural component to sustain significant inelastic deformation without considerable loss in strength. A single-anchor steel-to-concrete connection will behave in a ductile manner if its ultimate strength is controlled by the capacity of the steel. Alternatively, the ultimate strength of non-ductile single-anchor connections is controlled by the strength of the embedment [19,35]. However, multiple-anchor connections may suffer brittle failure if the most heavily loaded anchors fail before the capacity of the other connectors is reached.

2.3.2.2. Single-anchor connections in shear. Shear transfer across the interface of single-anchor steel-to-concrete connections is explained by two behavioral mechanisms: shear friction and dowel action (See Fig 2.5).

<u>Shear transfer by dowel action:</u> This mechanism of shear transfer across a shear plane is characterized by yielding and fracture of the dowels in bending, shear, or

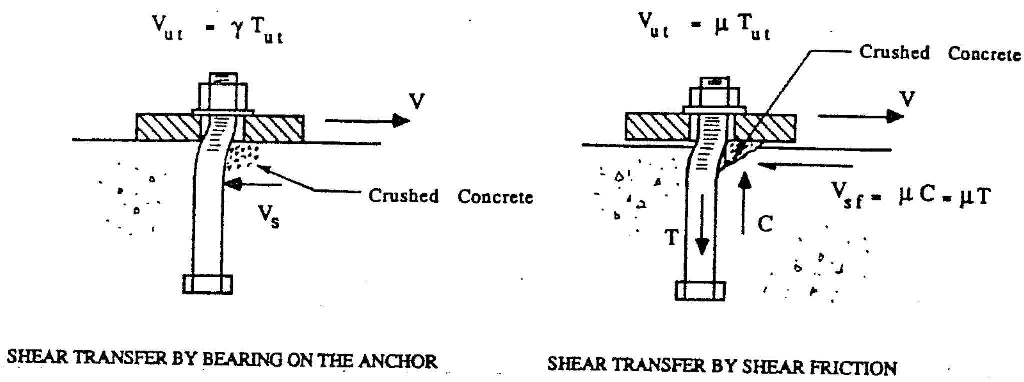


Figure 2.5 Anchor bolt shear transfer mechanisms [19]

kinking [36]. Shear is transmitted from the bolt to the concrete through bearing of the bolt at the exterior surface of the concrete which causes failure of the concrete on the loaded side of the anchor bolt. Concrete spalling may decrease the stiffness of the connection but does not limit its strength [13,19,24]. The characteristics of the shear transfer mechanism may differ depending on the fastening method used. For instance, as illustrated in Fig. 2.6, the behavior of welded studs is slightly different from the behavior of threaded rods because of the fixity provided by the weld near the steel-concrete interface.

Shear transfer by friction: This approach assumes that shear is transferred by a frictional force developed between the steel retrofit element and the concrete surface. Bearing of the bolt against the concrete forms a concrete wedge of approximately 1/4 the bolt diameter. Translation of the spalled concrete wedge under the shearing force produces a vertical thrust which causes tension in the anchor and consequently a frictional force at the wedge-plate interface (See Fig. 2.5) [5,13]. This behavioral model is used by ACI 349-85, which establishes that the capacity of a single-anchor steel-to-concrete connection is equal to the tensile force developed in the anchor multiplied by the shear friction coefficient between the two surfaces. This mechanism was not observed in

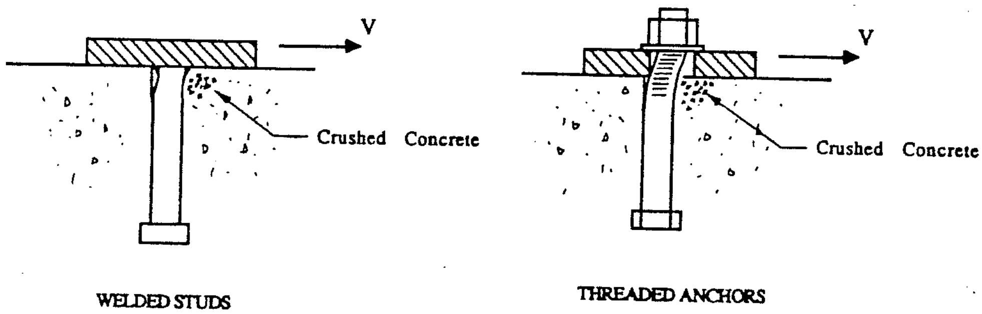


Figure 2.6 Deformation of welded studs and threaded anchors in shear [19]

Mendonca and Malik's tests [29], because the plate slipped away from the concrete wedge preventing the confinement required for the increase in friction to occur.

Shear friction versus dowel action: Experimental studies by Mattock et al. indicate that dowel action controls the mechanism of shear transfer when there is a preexisting crack in the interface plane [25,33]. Dowel action should therefore dominate behavior in the case of a steel element placed against existing concrete where the presence of an interface discontinuity is evident. This leads one to believe that the required coefficients of friction in ACI 349-85 reflect "apparent" coefficients of friction which include the effects of both frictional resistance and dowel action [19].

Shear transfer as described by Wiener: Based on multiple-fastener steel-to-concrete connection tests at The University of Texas at Austin [45], Wiener suggested that the shear transfer mechanism between steel and concrete was divided into three distinct stages:

1) Elastic stage: Shear transfer in this stage is characterized by friction (or adhesion if the plate is bonded) between the steel element and the concrete surface.

2) Post slip stage: Shear transfer is dominated by bearing of the anchor bolt against the surrounding concrete

3) Large deformation stage: Wiener observed an increase in frictional resistance near failure attributed to an increase in friction on the loaded side of the connection due to rotation of the anchor bolt which resulted in clamping of the nut and washer into the steel section.

2.3.2.3. Single anchor connections in combined shear and tension. Epoxy anchor bolts used to attach steel elements to existing concrete may fail as the result of combined shear and tension. The best fit to available data has been provided by a complex interaction equation provided by McMackin et al. [19]. A more conservative linear interaction equation has been proposed by Cannon [12].

2.3.2.4 Multiple-anchor connections in shear. Long joints behave differently than compact joints. Since load transfer is not distributed evenly among several connectors, the joint may suffer from sequential failure of the fasteners ("unbuttoning") before the retrofit plate attains its full strength [30,44]. This "long joint" effect is different depending on the section properties of the material joined, on the individual load-deformation response of the anchor bolts used, and on the spacing between connectors. To insure good performance of multiple-anchor connections, each anchor should achieve its shear strength prior to connection failure. Results reported on multiple-anchor steel-to-concrete connections have shown that shear forces redistribute near ultimate [22,24]. However, the ability to redistribute the load depends largely on the deformation capacity of the bolts. According to Fisher and Beedle, "deformation capacity, and not the departure from linearity, is the criterion that governs failure in a long joint [23]".

2.3.3 Issues regarding performance of bolted connections

2.3.3.1 Connection relaxation. Besides transferring shear, the purpose of steel-to-concrete connections in rehabilitation programs is to effectively clamp the steel plates and sections to the existing concrete members. The clamping level of a connection is improved by applying an initial preload to fasteners by torquing the nut. However, the difference between initial preload and the tension in a bolt in service must be distinguished. After the joint is assembled, the anchor bolt will suffer both short-term and long-term relaxation.

Short-term relaxation: In general, short-term relaxation occurs in bolted connections because some parts or components creep or flow after being loaded past their yield point [9]. Plastic deformation of high spots on the clamped surfaces (even if the bolt and components are properly installed, interface surfaces are never perfectly flat), and on soft portions of the threads induce short term relaxation.

Long-term relaxation: Long term relaxation of connections is mainly caused by corrosion, vibration, or creep [9]. In steel-to-concrete connections, long-term relaxation takes place due to creep in the load-transfer mechanism of retrofit anchors. Tests performed at the laboratories of Hilti, Inc. [7,42] have demonstrated that retrofit (expansion) anchors may lose over 50% of the initially applied clamping force over a two week period. The drop in the pretensioning force observed in those experiments was attributed to creep of the concrete in the zone where the expansion device is located.

When designing a steel to concrete connection, the engineer must be able to estimate the correct bolt loads through the life of the structure. Unfortunately, the author is not aware of any significant body of data on long-term relaxation of epoxy anchors.

Effect of bolt inclination: In rehabilitation projects, it is very unlikely that all anchors will be installed precisely perpendicular to the concrete surfaces. According to

Bickford [9], if the bolts are inclined, it may take a greater amount of tension to generate the necessary clamping force between joint elements, and as a result, relaxation will most likely be worse. Furthermore, when the steel element and bottom of the nut are not parallel, the bolt will not stretch uniformly and will bend as it is tightened. Bending alters the stress state within the bolt which may cause more plastic flow in the threads, and hence, greater short-term relaxation. Minor differences in connection geometry can result in substantial differences in the state of stress in the bolt.

Spring-loaded connections. To reduce connection relaxation, bolt preload may be transferred from the nut to the steel plate through a spring ("Beveled") washer as illustrated in Fig. 2.7. The spring between the plate and the washer controls the transfer of clamping force since it has a very low stiffness as compared to the other connection components. Because of the spring's low stiffness, a small deformation in the bolt will not make an appreciable difference in preload level. The disadvantages of using these components are that they are expensive, and manufacturers often have difficulty controlling the stiffness of the springs [9].

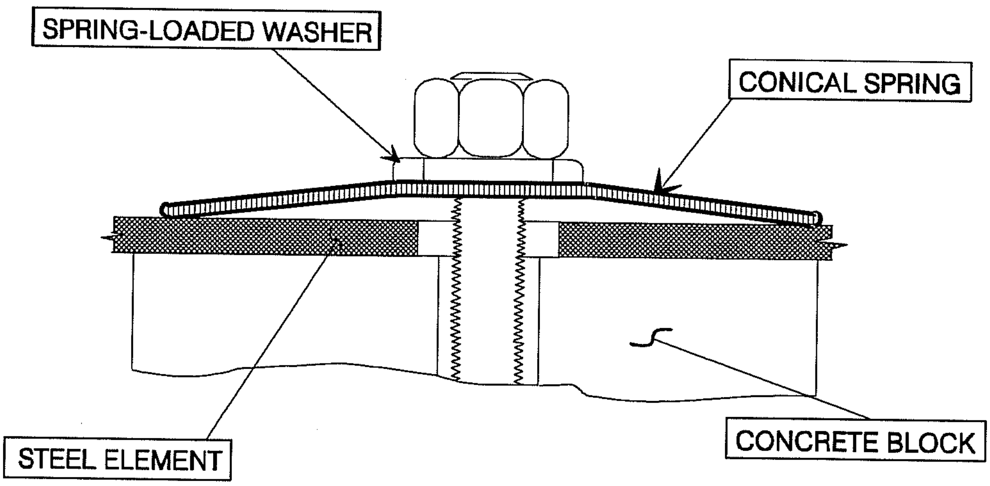


Figure 2.7 Spring-loaded connection

2.3.3.2 Oversize holes. Oversize holes are attractive for steel-to-concrete connections because they reduce the need for strict tolerances, alleviate misalignment problems, and allow for erection adjustments. However, the removal of plate material around the bolt causes two important problems:

1) the amount of slip necessary to bring the connection into bearing is increased, and

2) slip resistance is reduced.

The reduction in slip resistance is attributed to a loss of clamping force between the attached surfaces [3]. Reduced contact area between the washer and base plate results in dishing of the washer and reduced clamping force. Previous investigations on steel-to-steel connections showed an average decrease of 17% in the slip coefficient for 5/16 in. (instead of standard 1/16 in.) oversize holes [16].

2.3.3.3 Prying action. A further consideration in steel-to-concrete connections is prying action. The prying action mechanism is quite complex, and research on the subject is still being conducted [34]. Lift off of the steel strengthening elements due to tensile forces results in additional prying forces in the fasteners which are not easily determined. In addition, prying action may direct a large part of the total connection load to the most heavily loaded connectors [12,34].

Plate stiffness and bolt preload have a strong influence on prying action. Preload has a direct effect on the magnitude of load producing lift-off of the strengthening element from the concrete. Prying action is also reduced if the steel plates to be connected are stiff. The effect of plate flexibility should be taken into consideration if the distance between anchor bolts is greater than twice the thickness of the plate [12]. An approach to reduce prying action in steel jacketing rehabilitation projects is described in the next section.

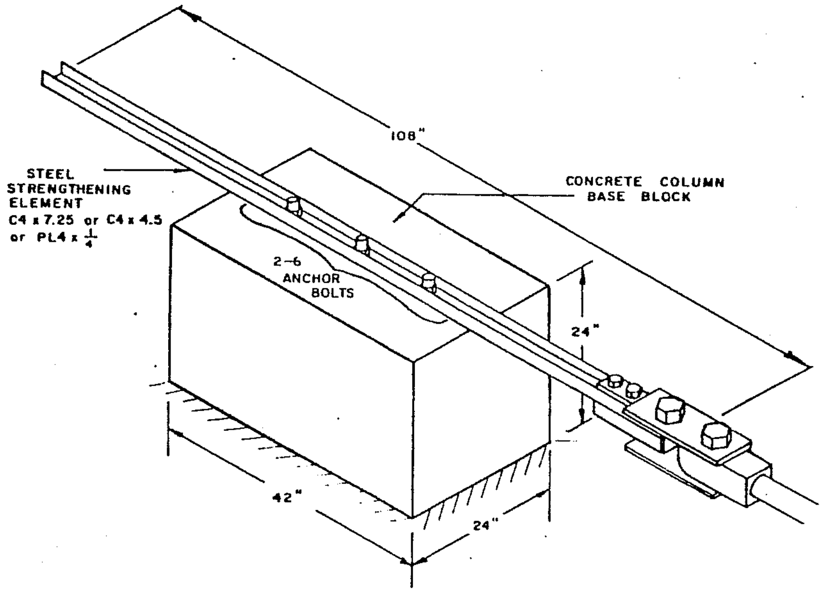


Figure 2.8 Test setup used in previous research on steel-to-concrete connections (Ref. 45)

2.4 Previous Research

An experimental investigation by Wiener at the University of Texas [45] studied the cyclic behavior of steel-to-concrete connections using epoxy-grouted anchor bolts. Wiener's test specimens consisted of a steel element attached to one side of a reinforced concrete block, simulating a portion of a concrete member (Fig. 2.8). The variables considered in this investigation included: number of fasteners, use or omission of epoxy at the steel-concrete interface, surface preparation, and steel element cross section. Envelope curves of load versus slip response of the six specimens tested by Wiener are shown in Fig. 2.9. As shown in the figure, epoxy at the interface improved elastic capacity by a factor of two, and reduced the amount of slip before capacity was reached. Epoxy inadvertently extruded into the void around each bolt formed a "bearing pad" which resulted in a more uniform force distribution among bolts, and hence, higher connection strength. The

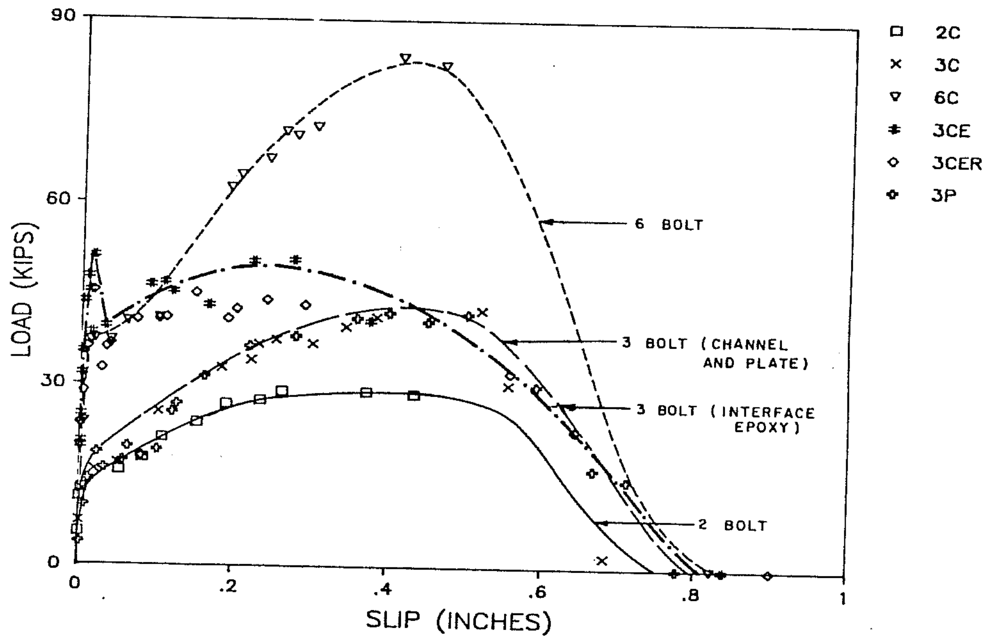


Figure 2.9 Envelopes of response in tests conducted by Wiener [45]

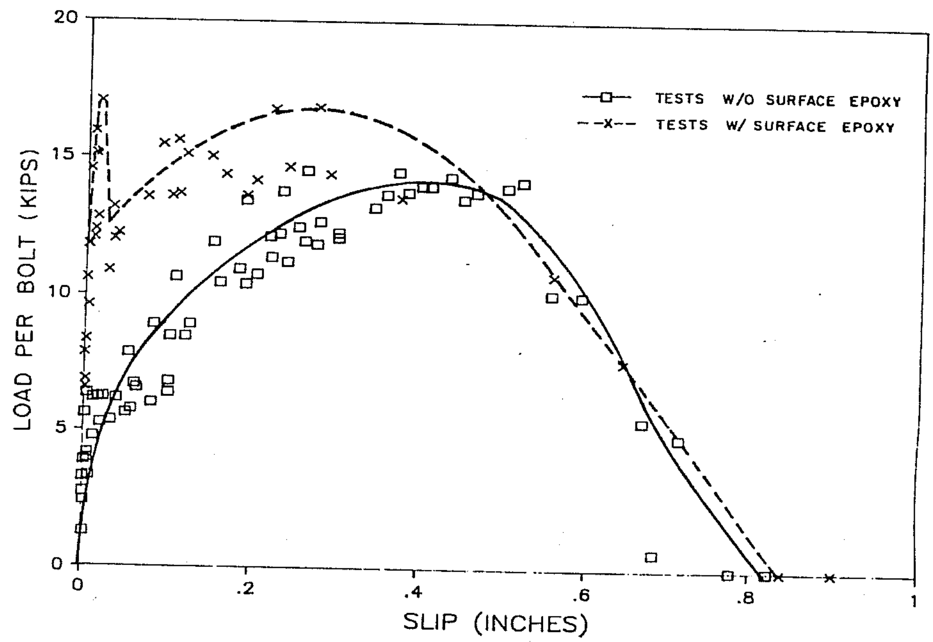


Figure 2.10 Average load per bolt versus slip in tests conducted by Wiener [45]

improved behavior of connections with epoxy is demonstrated in the envelope curve of average load per bolt versus slip illustrated in Fig. 2.10. Connection stiffness was clearly higher at all displacement levels. Tests of connections without epoxy in the interface showed substantial slip between steel and concrete. They also exhibited a greater non-uniformity of force distribution among the anchor bolts.

Following this investigation, Peeler studied the behavior of single-bolted connections with oversize holes incorporating epoxy adhesives to fill the void between the bolt and the holes in the steel plate [37]. Peeler tested seventeen specimens varying the hole clearance, the rod position in the hole through the steel element, and bolt preload. Regarding the use of epoxy fillers, Peeler concluded the following:

1) Adhesive fillers prevented major interface slip,

2) Connections with oversize holes filled with adhesive were stiffer, stronger, and more ductile than standard connections with no adhesive filler,

3) Position of the rod in the hole had no effect on the behavior of oversize holes filled with adhesive,

4) For connections filled with epoxy, as hole clearance increased, ultimate strength and deformation capacity increased but connection stiffness decreased,

5) Increased deformation capacity would result in a more uniform load distribution among anchor bolts in long connections

Peeler also recommended generous application of adhesive at the base of the rod to completely fill the void.

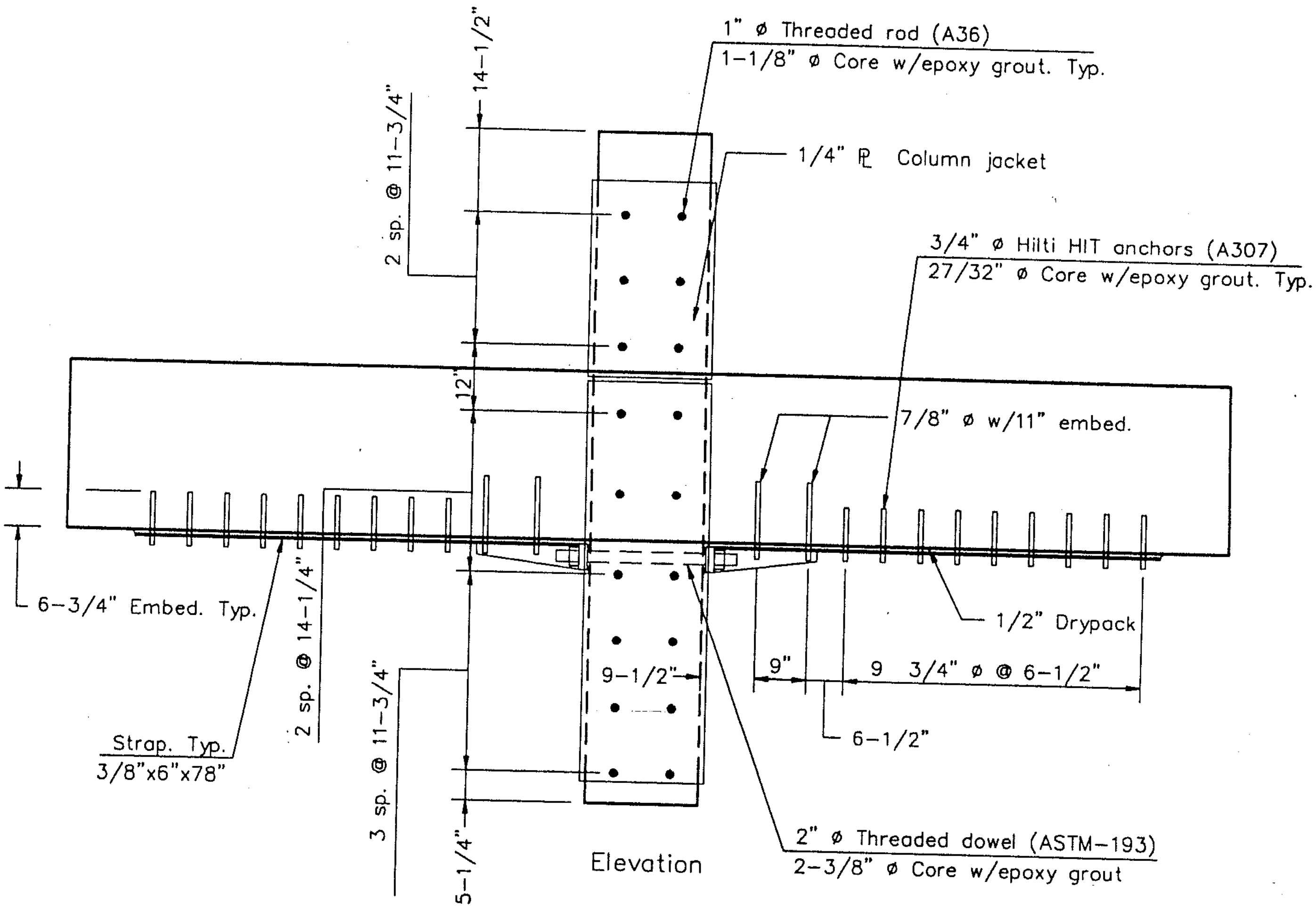


Figure 2.11 Details of the strengthening scheme tested by Estrada [22]

In a research program investigating the performance of a steel jacketing scheme for strengthening of a moment resisting concrete frame, Estrada reported excellent behavior of the steel-to-concrete connections [22]. Details of the strengthening scheme tested by Estrada are shown in Fig. 2.11. Shear was transferred from the existing concrete beam to the bottom steel strap through nine 3/4 in. diameter anchor bolts. The steel strap force was transferred through the column by the 2 in. diameter bolt which was anchored on the opposite face. A 1/2 in. thick layer of dry-packed non-shrink grout was placed between the strap and the beam. The bracket welded to the bottom steel strap was provided for continuity of reinforcement and also to insure development of the tensile capacity of the strap by reducing prying action for the connectors closest to the column [22]. The 7/8 in. anchor bolts used to connect the bracket to the beam had a deeper embedment length than for other bolts since they were expected to carry the tension generated by lift off of the plate. These bolts were placed through slotted holes so that no shear forces would be transmitted through them.

Results of tensile force distribution along the strap gave an indication of the loads

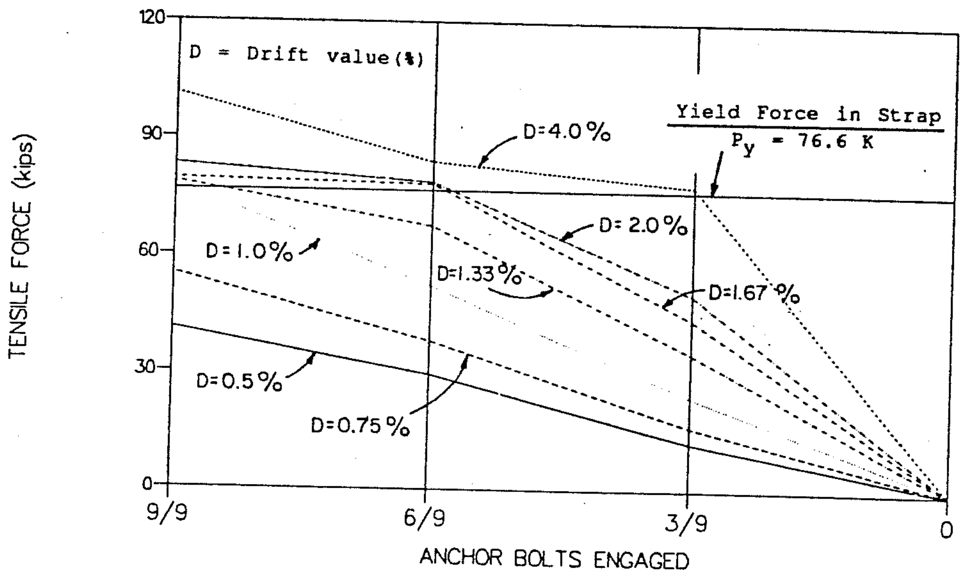


Figure 2.12 Load distribution in beam strengthening strap from test conducted by Estrada [22]

carried by each anchor bolt. As shown in Fig. 2.12, in early stages (up to 1% story drift) force transfer was distributed evenly, with each bolt carrying approximately 8.5 kips. However, a larger portion of the load was transferred by anchor bolts 1 through 3 after the strap yielded. The anchor bolts were able to sustain large deformations near failure in this experimental program because the space provided by the 1/2 in. thick grout layer allowed the bolts to bend in double curvature rather than deforming in direct shear across the interface plane.

CHAPTER 3
EXPERIMENTAL PROGRAM

3.1 Introduction

Thirty six tests were performed to investigate the behavior of single-anchor steel-to-concrete connections. The variables studied and the nomenclature used for test identification are explained. Material properties, specimen construction and instrumentation are described along with typical testing procedures.

3.2 Specimen Description

A schematic of a typical specimen is shown in Fig. 3.1. The specimen consists of a steel strengthening element fastened to a reinforced concrete block with a 3/4 in. epoxy-grouted anchor bolt. This configuration is intended to represent the attachment of a new steel member to existing reinforced concrete. The steel element was loaded in tension along its longitudinal axis to examine the shear transfer characteristics of the connection. Specimens were designed for failure of the anchor bolt, rather than failure by plate buckling or substantial concrete damage.

3.3 Test Variables

The variables studied in this project include:

1) Hole clearance between the anchor bolt and steel element

2) Clamping force provided by the tightened anchor bolt

3) Materials used to fill the annulus between the bolt and steel element (Fig. 3.2) and interface between the concrete block and steel element

4) Surface treatment of the steel element

5) Fastener components

6) Anchor bolt inclination

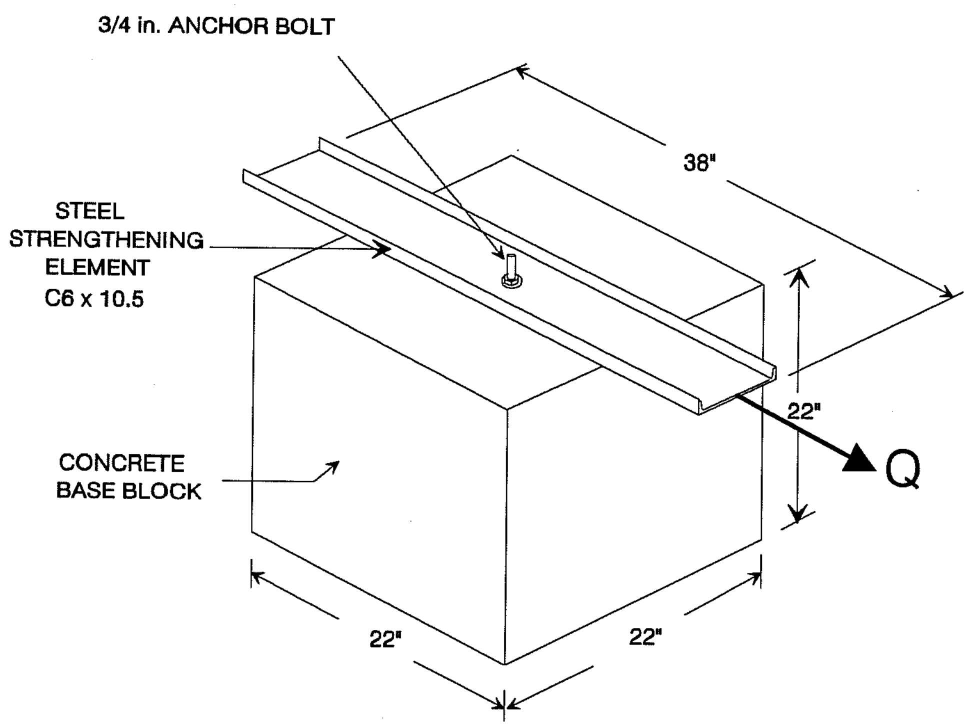


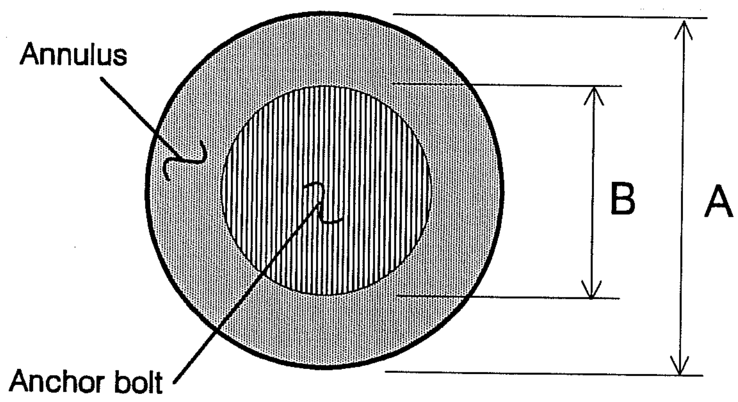
Figure 3.1 Schematic of test specimen configuration

7) Bolt position in the hole through the steel element

8) Thickness of interface material

9) Effect of reversed cyclic loads

More details related to the test variables are provided in the following sub-sections.



CLEARANCE= AMOUNT OF OVERSIZE= A-B

Where,

A = Diameter of hole in the steel element
B = Nominal diameter of the anchor bolt

Figure 3.2 Definition of "annulus", "clearance", and "oversize"

3.3.1 Hole clearance. Clearance refers to the difference in hole diameter in the steel element and nominal diameter of the anchor bolt, as shown in Fig. 3.2. Two hole clearances were considered:

1) 3/16 in.

2) 7/16 in.

A 3/16 in. clearance was used in previous research on steel-to-concrete attachments. It is the maximum allowed by the AISC design specification for slip-resistant steel connections with 3/4 in. diameter bolts [6].

The use of holes with 7/16 in. clearance is appealing because it provides greater tolerance for erecting steel after bolts have been installed.

3.3.2 Clamping force. Three levels of clamping were studied:

1) 12 kips clamping (60% of yield in the anchor bolt)

2) 18 kips clamping (90% of yield in the anchor bolt)

3) Minimal clamping, applied by hand tightening the nut

Different levels were selected because the clamping force is so important in the connection, and there is concern about long term anchor relaxation due to creep in the epoxy and/or surrounding concrete. The effect of hand tightening was included to clarify what happens if the clamping force is initially low, or if it is lost due to relaxation.

The force was estimated using a Skidmore machine which gives the bolt force for a given torque applied to the nut. Using the average for three different fasteners, it was determined that torques of 140 and 200 foot-pounds would yield clamping forces of 12 and 18 kips, respectively. No adjustments were made in the torque applied to the nuts in tests with inclined anchor bolts. As a result, the magnitude of the applied clamping force is less certain than for tests with a "straight" anchor.

3.3.3 Filler and interface materials. The following types of interface materials were used in selected test specimens:

1) High modulus epoxy adhesive, and

2) Non-shrink grout.

The role of these materials in improving connection behavior was studied from two perspectives:

1) Role of the materials as a concrete-steel interface agent improving friction or adhesion capacity, and

2) Ability of the material to completely fill the void between the anchor bolt and hole in the steel element and improve the characteristics of load-slip response.

3.3.4 Surface treatment. The effect of surface preparation for the steel on load at first slip was studied. Behavior of specimens with surfaces prepared with light sandblasting was compared with acetone-cleaned surfaces.

3.3.5 Fastener components. Three methods were used hold the steel element in place:

1) Hexagonal nut and mild steel washer,

2) Hexagonal nut with a spring-loaded washer assembly,

3) Field welding the nut to the steel element.

Spring-loaded washers are attractive because they have the potential of reducing long term relaxation effects in the connection. As seen in Figs. 3.3 and 3.4, the spring-loaded washer assembly is composed of two different parts: two flexible conical washers which act as a spring between the nut and steel channel, and a stiff disc that fits between the conical washers and the nut. Two different sets of conical washers were used in tests to provide different clamping forces and to permit direct comparisons with results from specimens with conventional fasteners having different bolt preloads (See Fig. 3.4).

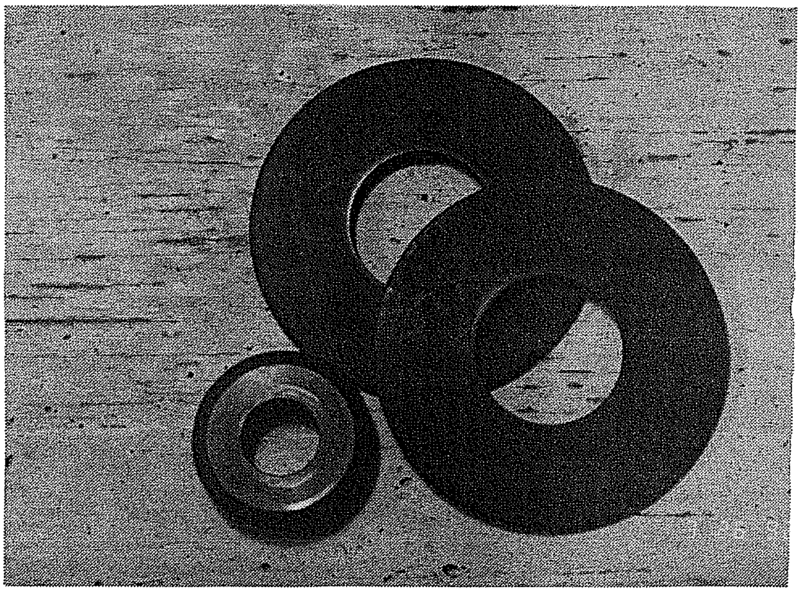
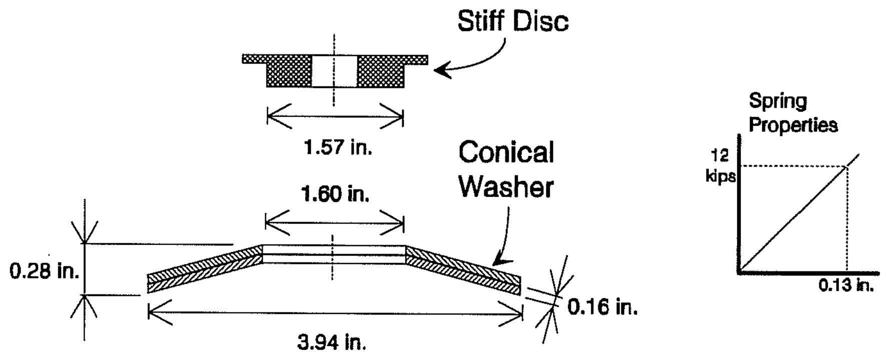
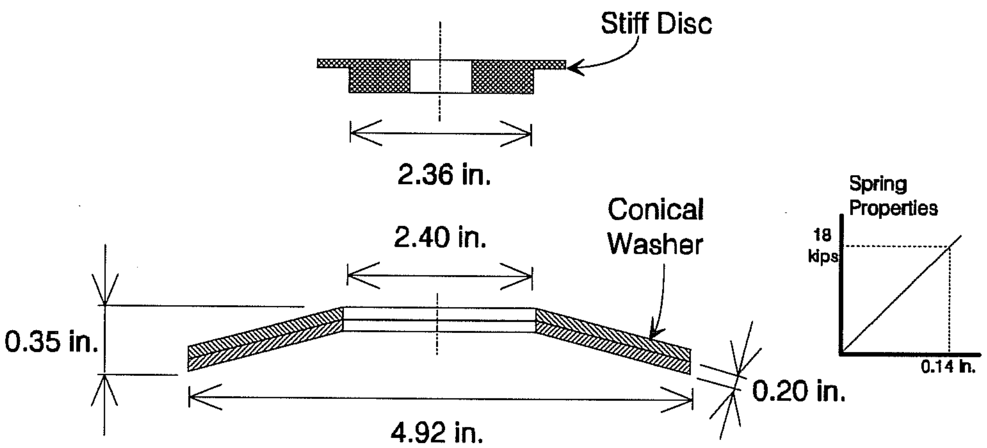


Figure 3.3 Spring-loaded washer components

Behavior of steel-to-concrete connections was anticipated to be improved by welding the nut to the steel element, as illustrated in Fig. 3.5. Attaching the nut to the strengthening element will preclude slip between the element and the anchor bolt and will provide immediate engagement of the bolt, making connection behavior less dependent on clamping force.



(a) Case A: 12 kips clamping force



(b) Case B: 18 kips clamping force

Figure 3.4 Schematic of details and load-deformation relationships for spring-loaded washers (according to manufacturer)

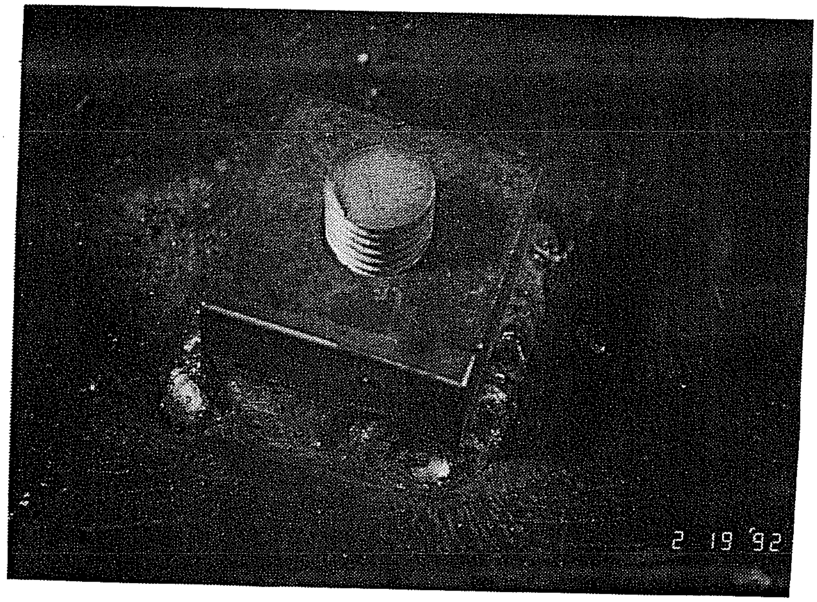


Figure 3.5 Nut welded to steel element

3.3.6 Bolt inclination. Initial bolt inclination due to construction tolerances and errors is a phenomenon commonly encountered in practice (See Fig. 3.6). Three monotonic tests were performed with the anchors inclined in different directions with respect to the direction of loading (See Fig. 3.7). A cyclic test with the bolt initially positioned leaning backward was also performed. The inclination of the anchors with respect to a plane normal to the concrete surface in this series of tests is indicated in Tables 3.1 and 3.2.

3.3.7 Bolt position. Behavior of a specimen with an epoxy-filled annulus and with the anchor bolt positioned on the side of the hole was compared to a companion specimen with the bolt located at the center. Previous tests showed that bolt position does not affect connection behavior if the annulus is filled with epoxy [37]. However, no tests in that series were performed with the bolt located on the side of the hole.

3.3.8 Interface thickness. All specimens, except one, incorporating non-shrink

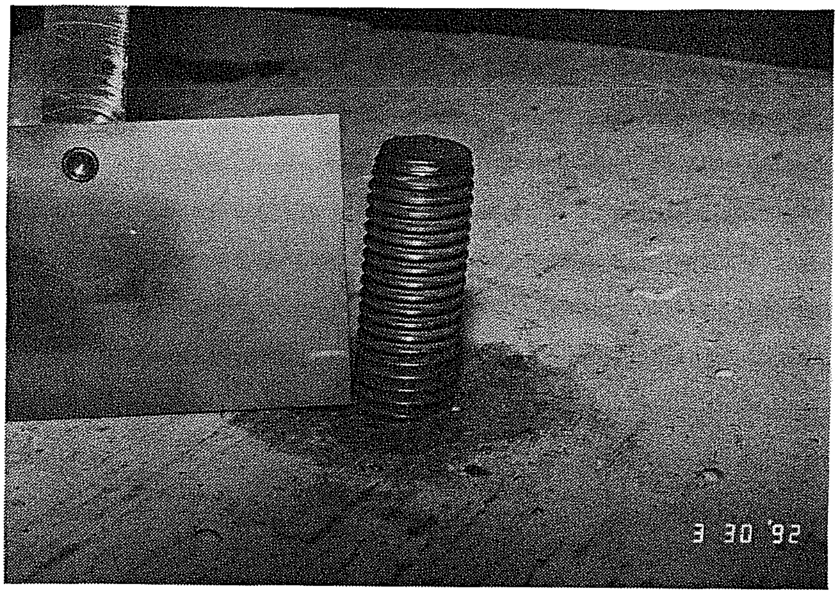


Figure 3.6 Anchor bolt with initial inclination

grout had an interface thickness of 1/4 in. One specimen having a 1/2 in. thick interface was included to study the differences in behavior introduced by this change. Varying thicknesses are often encountered in practice due to mismatched surfaces. Furthermore, connections in a previous steel jacketing research project having an interface thickness of 1/2 in. exhibited ductile behavior with the anchors deforming in a combination of shear and bending [22].

3.3.9 Repeated cyclic loading. Of the 36 specimens, 22 were loaded monotonically until failure and 14 were tested cyclically. The effect of reversed cyclic loads on the load-deformation relationship of the connections is obtained. Behavior during cyclic tests can be compared with monotonic behavior through comparisons of response envelopes.

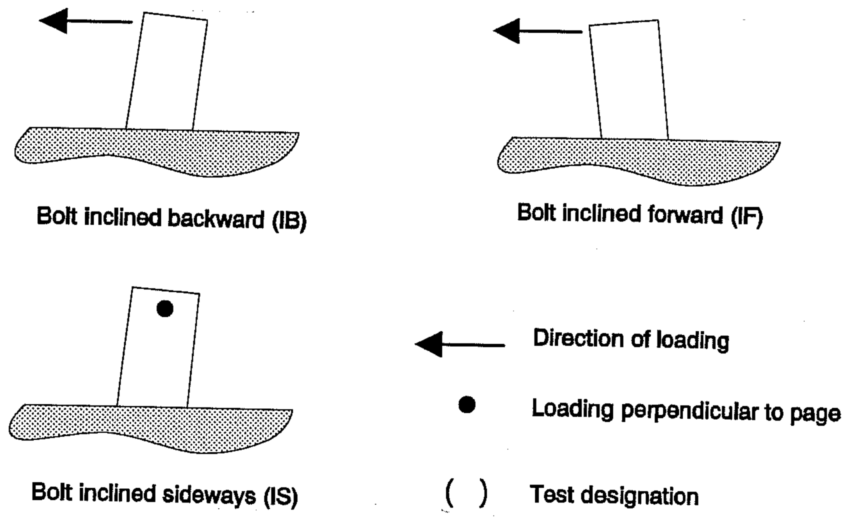


Figure 3.7 Position of anchor bolts in inclined-anchor tests

3.4 Specimen Designation

Each specimen was identified as shown in Fig. 3.8 by the following symbols: 1) A letter corresponding to the type of test, 2) A single letter describing the type of filler material and how it was applied, 3) A number identifying the hole clearance used, 4) A letter identifying the clamping force applied, and 5) One or two letters, following a hyph en indicating any other additional features.

Replicate tests are differentiated by a number immediately preceding the name of the test and separated by a hyphen. For example 2-Me3t would indicate the second monotonic test of a connection with a 3/16 in. oversize hole, epoxy in the annulus, and the nut tightened to a 140 foot-pound torque (to provide a 12 kip clamping force).

Designations and details of each test in the experimental program are listed in Tables 3.1 through 3.3.

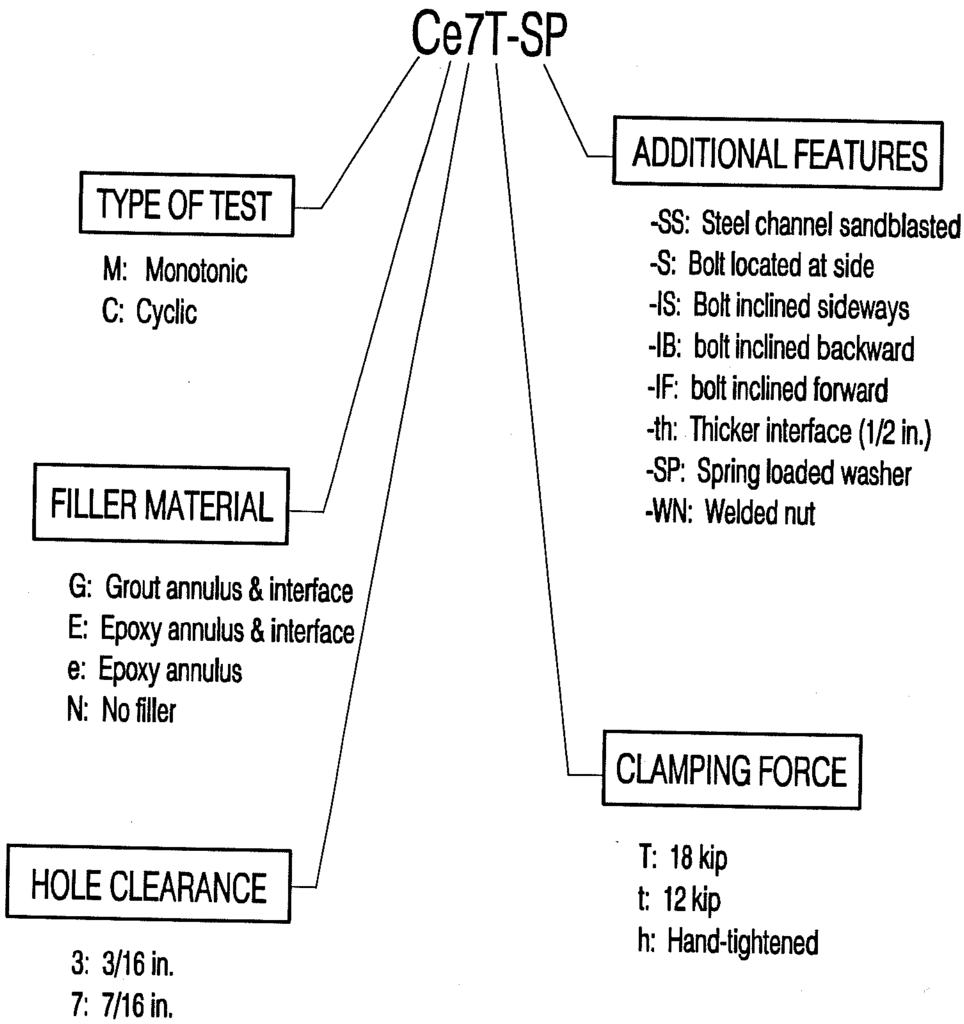


Figure 3.8 Test designation key

Table 3.1 Description of specimens incorporating epoxy adhesive

TEST	Base Block		Steel Section		Anchor bolt	
	f'c @ test date (psi)	Surface treatment	Hole oversize (in)	Surface tratment	Preload (kip)	Inclina-tion (degrees)
Me3h	4005	---	3/16	acetone	snug tight	---
ME3h	4005	---	3/16	acetone	snug tight	---
2-ME3h	4005	sandblast	3/16	acetone	snug tight	---
Me3t	4130	---	3/16	acetone	12	---
Me3t-S	4130	---	3/16	acetone	12	---
Me7h	4130	---	7/16	acetone	12	---
Me7t	4130	---	7/16	acetone	12	---
ME3h-SS	4130	sandblast	3/16	sandblast	snug tight	---
ME3t-SS	4130	sandblast	3/16	sandblast	12	---
Me3t-IS	4670	---	3/16	acetone	12	4 -Side
Me3t-IB	4670	---	3/16	acetone	12	5 -Back
Me3t-IF	4670	---	3/16	acetone	12	5 -Front
Me3T	4700	---	3/16	acetone	18	---
Me7T	4700	---	7/16	acetone	18	---
Ce3t	4700	---	3/16	acetone	12	---
Ce3T	4700	---	3/16	acetone	18	---
Ce7t	4700	---	7/16	acetone	12	---
Ce3t-SP	3470	---	3/16	acetone	12	---
Ce7t-SP	3470	---	7/16	acetone	12	---
Ce3T-SP	3470	---	3/16	acetone	18	---
Ce7T-SP	3470	---	7/16	acetone	18	---
2-Ce7t-SP	3470	---	7/16	acetone	12	---
Ce3t-IB	3470	---	3/16	acetone	12	5-Back
2-Me3t	3470	---	3/16	acetone	12	---

Table 3.2 Description of specimens incorporating non-shrink grout

TEST	Base Block	Steel Section		Grout Interface		Anchor bolt
	f'c @ test date (psi)	Hole oversize (in)	Surface treatment	Grout strength (psi)	Thickness (in)	Preload (kip)
MG3t	4005	3/16	acetone	5480	1/4	12
MG3t-SS	4670	3/16	sandblast	7490	1/4	12
MG7t-SS	4670	7/16	sandblast	7490	1/4	12
MG3t-th	4670	3/16	acetone	7490	1/2	12
MG7T-SS	3450	3/16	sandblast	7880	1/4	18
CG3t-SS	3450	3/16	sandblast	7880	1/4	12
CG7t-SS	3450	7/16	sandblast	7880	1/4	12
CG3T-SS	3450	3/16	sandblast	7880	1/4	18
2-CG3T-SS	3450	3/16	sandblast	7880	1/4	18
CG7T-SS	3450	7/16	sandblast	7880	1/4	18

Table 3.3 Description of specimens without filler materials

TEST	Base block	Steel Section		Anchor bolt
	f'c @ test date (psi)	Hole Oversize (in)	Surface treatment	Preload (kip)
MN3t	4005	3/16	acetone	12
CN3h-WN	3450	3/16	acetone	snug tight

3.5 Materials

3.5.1 Concrete. Three casting operations were performed with six blocks being cast at a time. The mix proportions were slightly different for each casting because concrete batches were shared with other repair and strengthening projects in the laboratory. All three proportions and average cylinder strengths at 28 days for each mix are given in Table 3.4. The concrete compressive strength for each specimen at the time of testing is included in Tables 3.1 through 3.3.

Table 3.4 Concrete mix proportions and strengths at 28 days

Component	Mix Quantities (pounds per cubic yard)		
	Mix A (08/27/91)	Mix B (10/10/91)	Mix C (12/16/91)
Cement	378	445	360
Coarse Aggregate	1880	1620	1890
Fine Aggregate	1442	1710	1498
Water	136	150	135
28-day Strength (psi)	3519	4436	3160

3.5.2 Threaded rod. Anchor bolts were cut from 3/4 in. diameter mild steel (ASTM A36) threaded rod. An 18 in. long sample of the rod was used as a tensile coupon. The load-deformation curve obtained for the 3/4 in. diameter threaded rod is shown in Fig. 3.9. Yield and fracture occurred at 59 ksi and 76 ksi respectively, based on the AISC tensile stress area [6,39].

3.5.3 Adhesive. The adhesive used at the interface and/or in the annulus was Sikadur 31 Hi-Mod Gel, a 2-component, solvent-free, moisture insensitive, structural epoxy, with a compressive modulus of elasticity of 830 ksi. It has a specified 7-day compressive

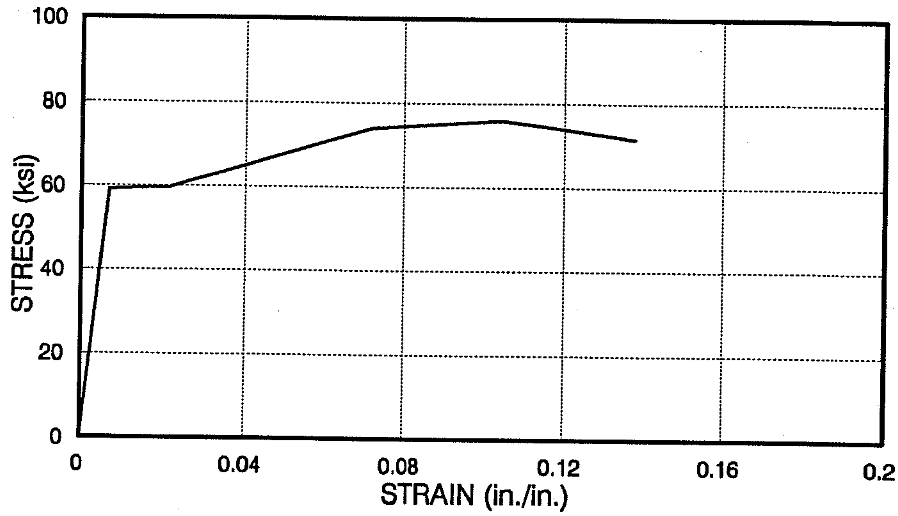


Figure 3.9 Stress-strain curve for threaded rod

strength of 12000 psi and a 14-day shear strength of 3400 psi at 70 degrees F. According to the manufacturer, this epoxy gains 75% of its ultimate strength after 16 hr. at 73 degrees F. The pot life of this product is 30 minutes at 70 degrees F.

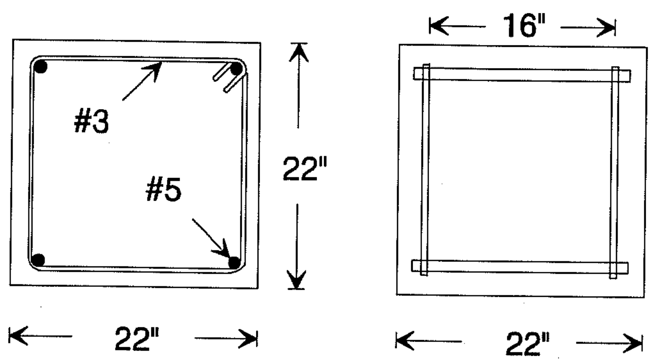
3.5.4 Non-shrink grout. The grout used at the interface and in the annulus of some connections was a pre-mixed non-shrink grout meeting Corps of Engineering Specification CRP-C 621 and ASTM C 1107. Grout compressive strength at the time of testing for each specimen is shown in Table 3.2, and was estimated from the average of three 3 x 6 in. cylinder tests.

3.6 Construction of Specimens

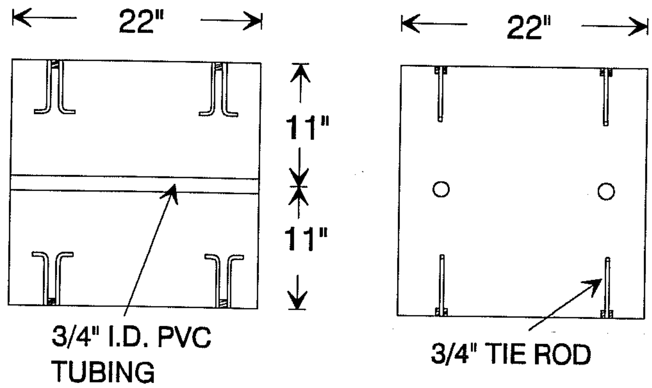
3.6.1 Base block construction. All blocks were cubes measuring 22" on each side, as shown in Fig. 3.10. Two test specimens were prepared from opposite faces of each concrete block.

Block reinforcing details are illustrated in Fig. 3.10. Number 5 deformed steel bars at the corners were tied by number 3 bars spaced at 16 in. A minimum of 1 in. cover was provided on all sides.

Formwork was constructed using 3/4 in. AC plywood with 2 x 4 in. bracing.



(a) Block reinforcement



(b) Block insert detail

Figure 3.10 Base block reinforcement and insert details

Forms were lacquered and oiled prior to assembly and then tied together with 3/8 in. threaded rod and lag screws. Lifting inserts and PVC tubing were embedded in each block (Fig. 3.10-b and 3.11) to facilitate handling and installation of the specimens into the test frame. Completed formwork is shown in Fig. 3.12.

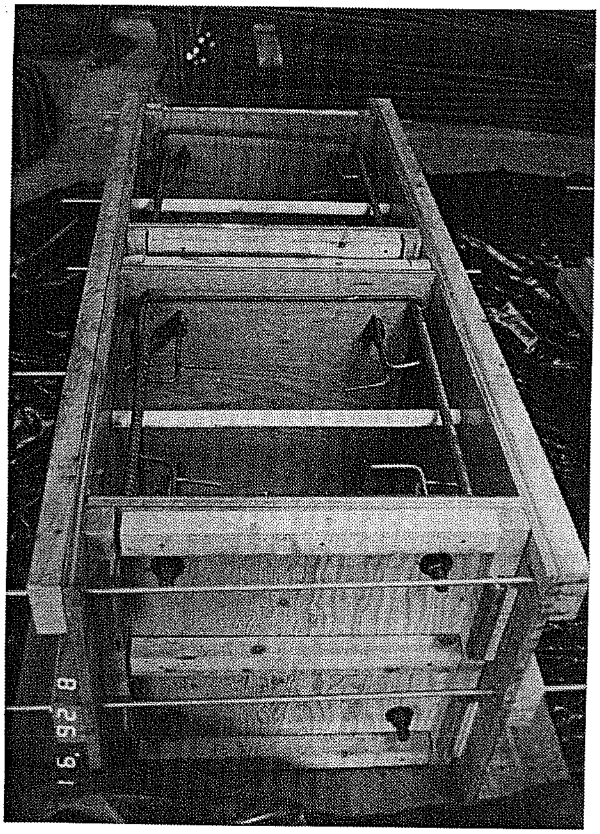


Figure 3.11 Formwork with reinforcement and inserts in place

3.6.2. Steel strengthening elements. The strengthening elements used consisted of A36 C6 x 10.5 steel channel sections. Either a 15/16 or a 1-3/16 in. diameter hole was drilled into the web of each channel to attach the steel element to the concrete member with the 3/4 in. anchor bolt. Figure 3.13 shows typical machining details for the steel sections used.

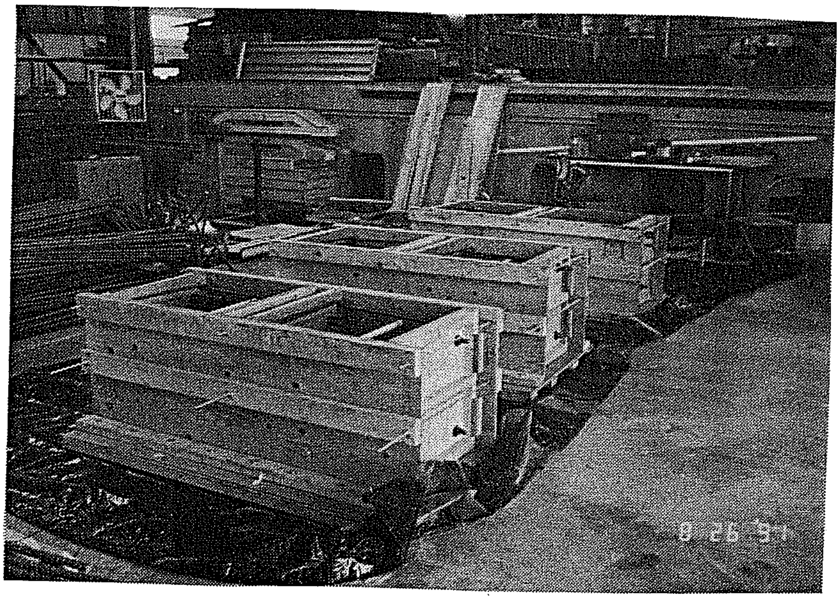


Figure 3.12 Completed formwork

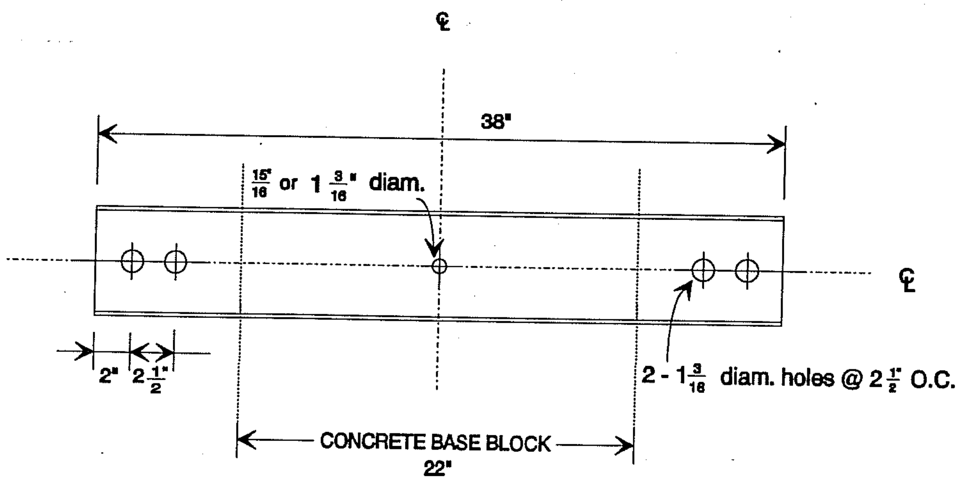


Figure 3.13 Schematic of steel element machining details

3.6.3. Anchor bolts. Bolts were brush cleaned prior to installation to ensure good bond. The anchors were embedded to a depth of 8 in. and extended a maximum of 2 in. above the concrete surface to allow the use of a socket wrench when later tightening the nut.

All holes were drilled after the specimens had been field cured for 28 days. Holes were drilled with an electric rotary percussion drill with a 27/32 in. diameter bit. A wooden guide was used when the anchor had to be perpendicular to the concrete surface (See Fig. 3.14) The holes were brushed with a bottle brush and vacuumed after drilling to remove loose concrete particles.

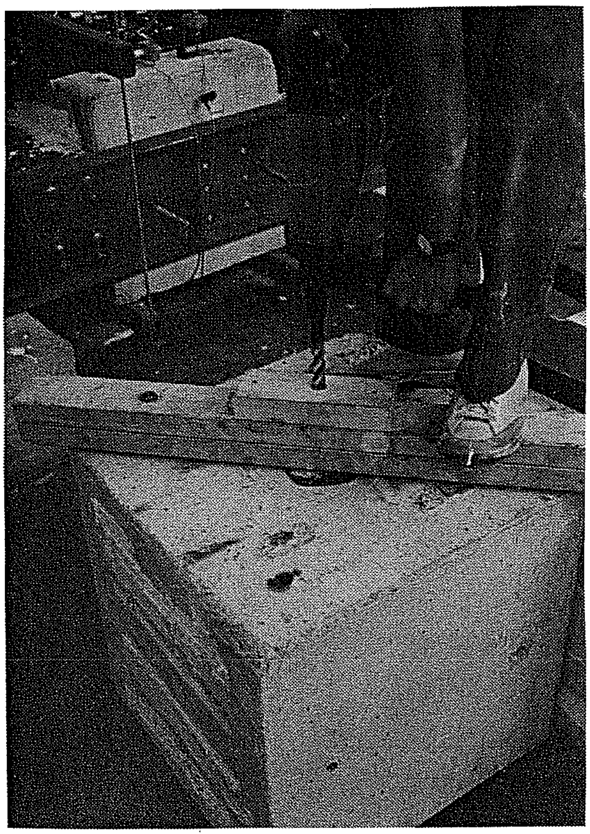


Figure 3.14 Concrete drilling with wooden guide in place

For specimens in which the effect of anchor inclination was to be studied, holes were drilled at a slight angle without the use of a guide. Furthermore, the bolts were pushed to the side of the hole as shown in Fig. 3.15 to enhance their inclination.

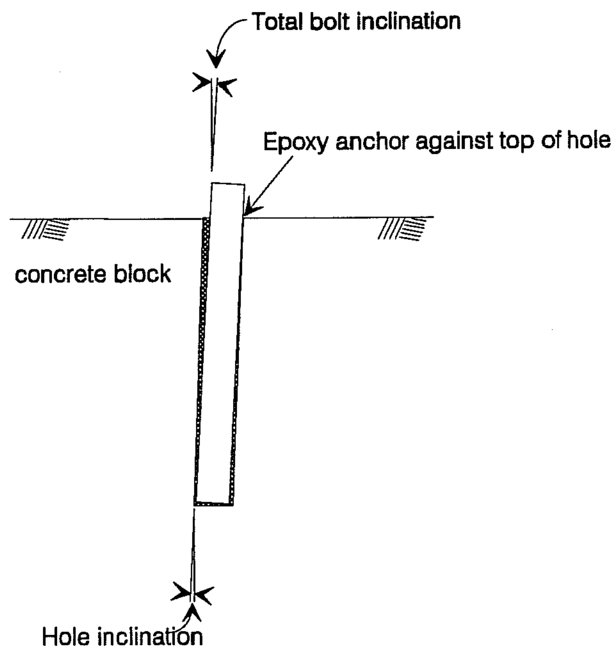


Figure 3.15 Schematic of components contributing to bolt inclination

Epoxy used to grout the anchor bolts was supplied in a prepacked cartridge as shown in Fig. 3.16. The epoxy is a two component bonding agent with a gel time of 4 min. at 68 degrees F, and a curing time of 45 min. at 68 degrees F. The cartridge is placed into an injector "gun" and the two components mix in the nozzle as they are squeezed from the cartridge. Holes were filled 40% with adhesive prior to bolt installation. The anchor bolt was set by rotating it while pushing it into the hole to ensure good coating with adhesive. Excess epoxy was removed from the surface and final alignment adjustments were made before the epoxy hardened. Figure 3.17 shows the final product of this operation.

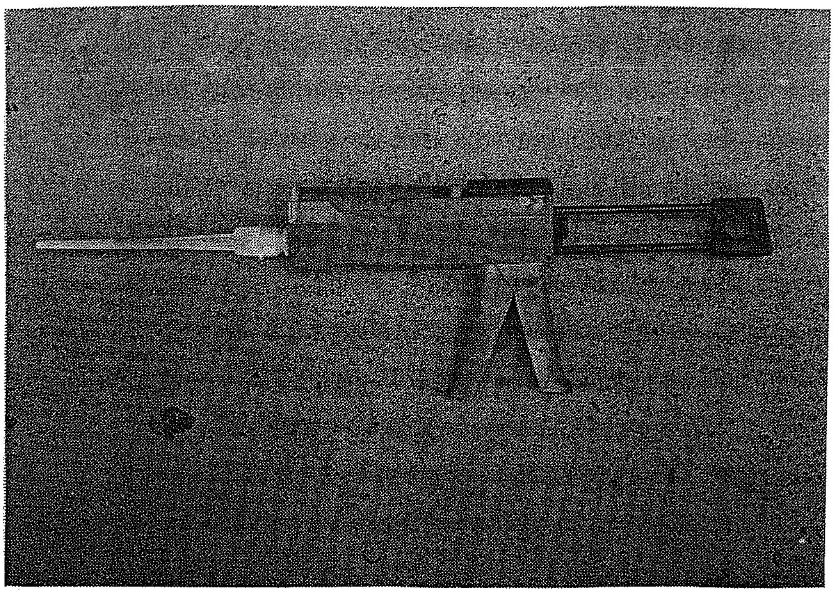


Figure 3.16 Two-component cartrige and nozzle in applicator

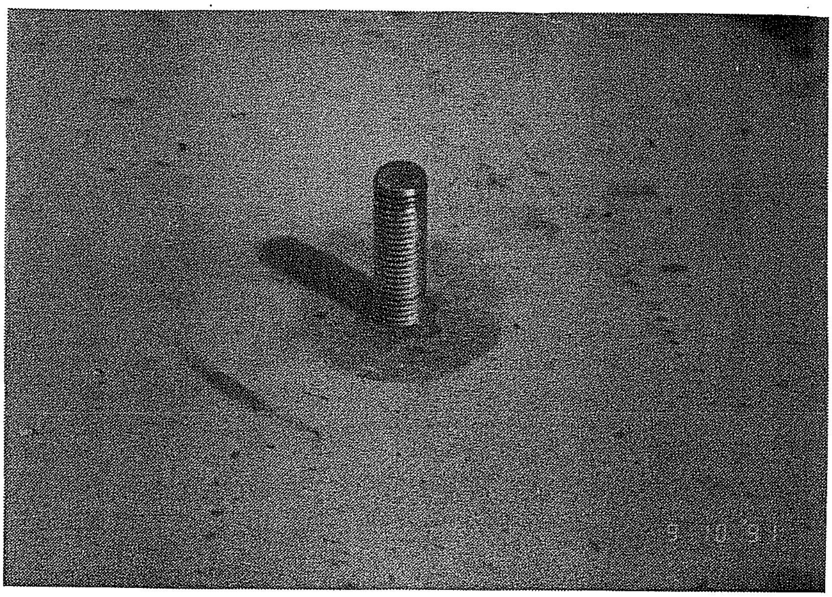


Figure 3.17 Bolt in place after removal of excess epoxy

3.6.4. Surface preparation.

3.6.4.1. General. Excess epoxy around the threaded rod was chipped to minimize surface irregularities. For all specimens, the concrete surface was cleared of dust and loose material using oil-free compressed air before placing the steel element. Furthermore, the surfaces of the channel sections were wiped clean with acetone.

3.6.4.2. Specimens requiring sandblasting. As indicated in Tables 3.1 through 3.3, light sandblasting was applied to roughen the interface surface of each concrete block in tests with interface epoxy or non-shrink grout (Fig. 3.18). The contact surface of the steel elements may or may not have been sandblasted. Sandblasting removed the fine aggregate up to a depth of approximately 1/16 in. in the concrete, and removed the mill scale on the steel sections.

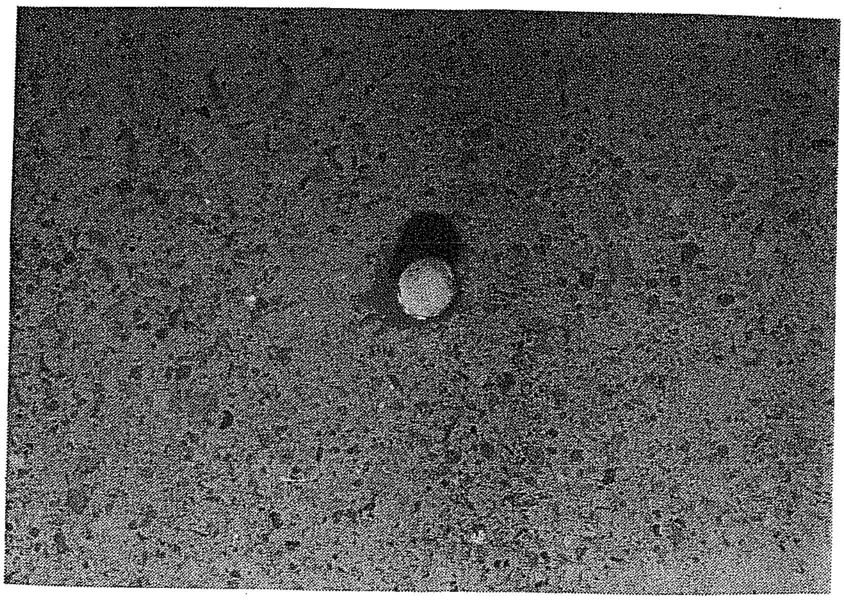


Figure 3.18 Sandblasted concrete surface

3.6.5. Adhesive application.

3.6.5.1. Specimens with interface epoxy. Procedures recommended by the manufacturer were closely followed during mixing and application. Epoxy was applied to the concrete surface over an area of 6 x 12 in. (Fig. 3.19). A bead of silicone caulk was placed at the ends of the area to maintain the length of the interface contact surface. The channel section was pushed over the epoxy interface and the fastener was installed and tightened, which squeezed the interface epoxy to a thickness of 1/32 to 1/16 in. Excess adhesive extruded along the sides of the channel was removed and each specimen was allowed to cure for a minimum of seven days.

Figure 3.19 Epoxy adhesive in place over concrete surface

3.6.5.2. Specimens with epoxy in the annulus. Adhesive was generously applied to the base of the anchor bolt and surrounding concrete (See Fig. 3.20). This procedure insured that the void between the bolt and the hole was completely filled when the channel was placed (Fig. 3.21). The steel section was held in the right position using wooden wedges as illustrated in Fig. 3.22. The washer was then placed on top of the extruded epoxy and the nut was tightened. Epoxy on the threads did not interfere with the efficiency of the tightening procedure. Some bonding at the interface due to extrusion of excess epoxy was expected.

3.6.5.3. Specimens with a spring-loaded washer. A similar procedure as for specimens with epoxy in the annulus was followed, except that excess adhesive extruded out of the hole was removed prior to placing the conical washers. Therefore, there was no bonding between the spring loaded device and the steel channel. An installed spring-loaded connection is shown in Fig. 3.23.

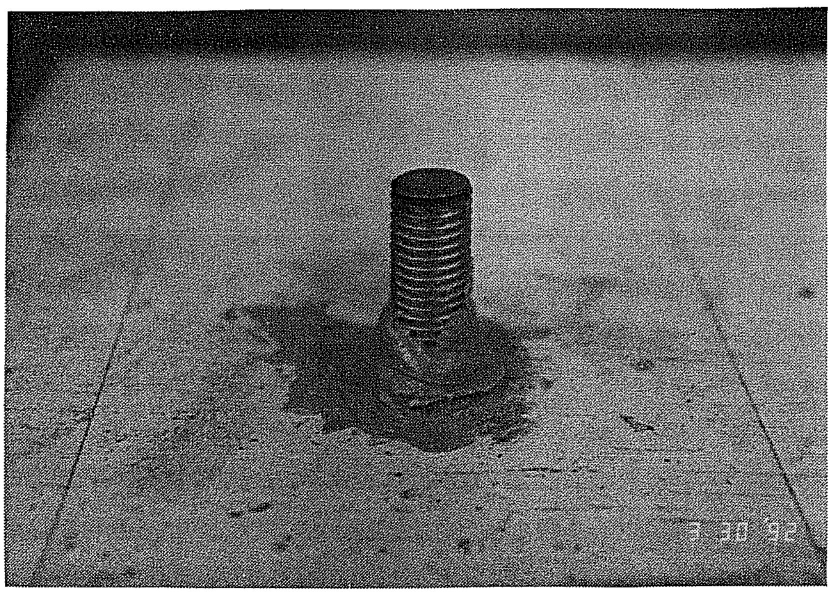


Figure 3.20 Epoxy applied to base of anchor bolt

Figure 3.21 Channel in position with extruded adhesive

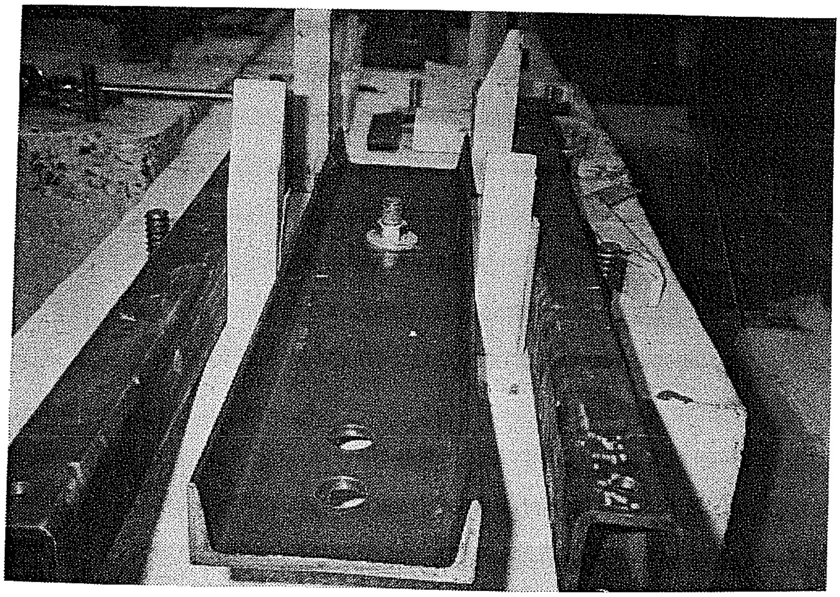


Figure 3.22 Channel held in position with wooden wedges during curing

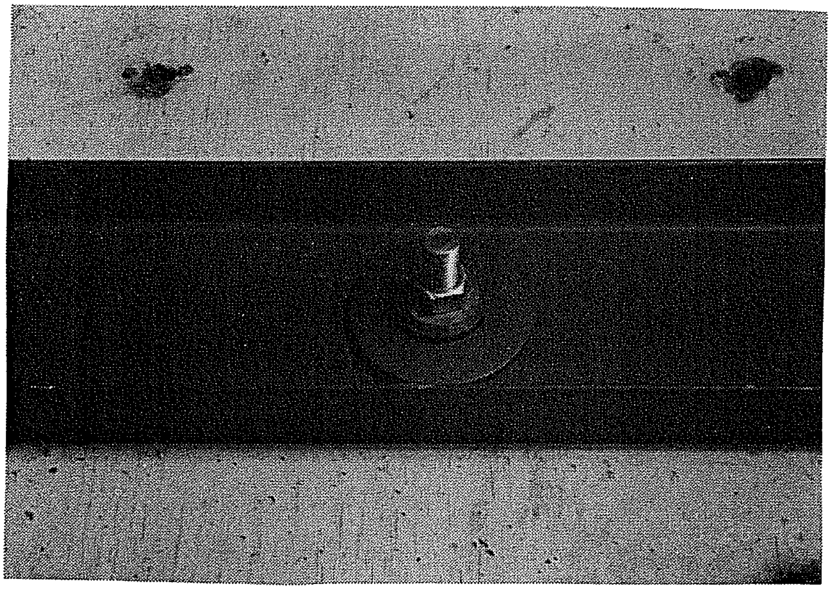


Figure 3.23 Spring-loaded washer assembly

3.6.6. Non-shrink grout application. Non-shrink grout was prepared as recommended by the manufacturer. One and one-half gallons of water per 55 lb. bag were mixed in a paddle-type mortar mixer. Grout was applied over an area of 12 x 6 in. with a trowel after the concrete block and steel surfaces were prepared. The length and thickness of the grouted interface were controlled by placing plywood spacers of appropriate thickness at the ends of the interface area. Excess grout extruded on the sides of the channel and through the hole was removed after the steel section was positioned (Fig. 3.24 and 3.25). Specimens were allowed to cure for a minimum of fourteen days before testing. No special curing procedures were implemented.

3.6.7. Anchor bolt tightening procedure. All specimens with epoxy were tightened immediately after placement of the steel section. The nuts were installed and tightened to the required tension using a torque wrench. Hand-tightened bolts were simply "snugged" using a 12 in. wrench.

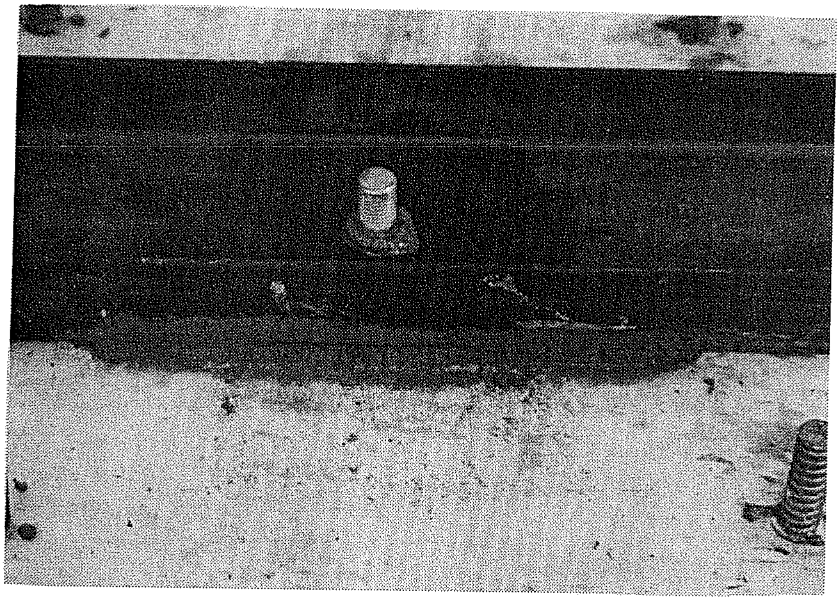


Figure 3.24 Channel placed over applied grout

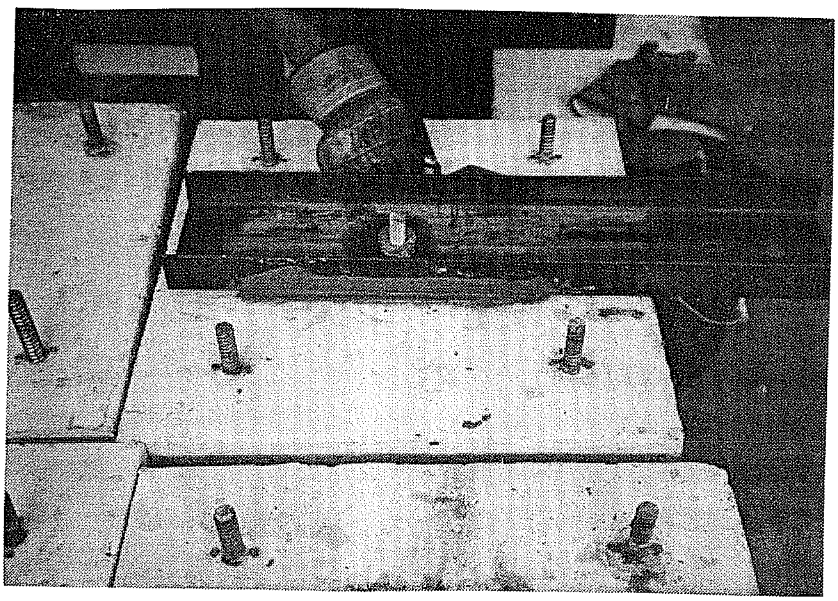


Figure 3.25 Specimen during removal of excess grout at interface

All bolts in specimens with grout were carefully brush cleaned then tightened fourteen days after grout application.

3.6.8. Welded nut test. The nut used in this test was cut from a piece of 1/2 in. A36 steel plate and was machined as illustrated in Fig. 3.26. The nut was hand-tightened and then welded to the steel channel as shown in Fig. 3.5.

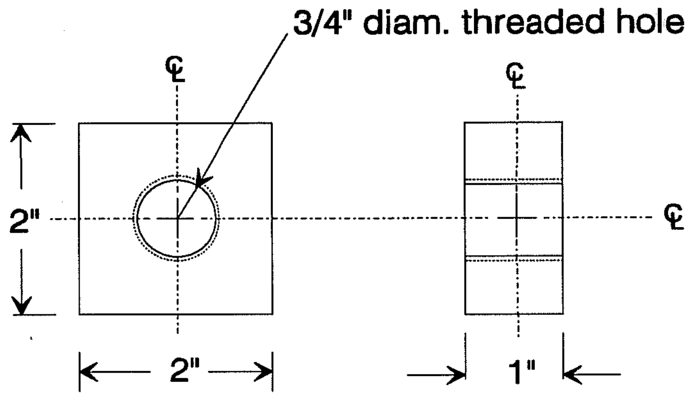


Figure 3.26 Dimensions of weldable nut

3.7. Test Frame

The testing frame illustrated in Fig. 3.27 through 3.30 was designed to insure loading of the steel element in tension, parallel to the interface plane. The frame was fabricated from two 12-in. channels with 1/2 in. stiffeners. The channels were spaced at 24 in., two inches more than the width of each specimen, for ease of installation and to allow for imperfections. Two 10-in. deep restraining girders were positioned at each end of the test specimen to prevent movement of the base block during testing. Hydraulic centerhole rams, provided at the ends of the frame were used to load the strengthening element. Rods passing through the rams were attached to the steel element by adjustable hinged end connections that directed the load through the channel's neutral axis in order

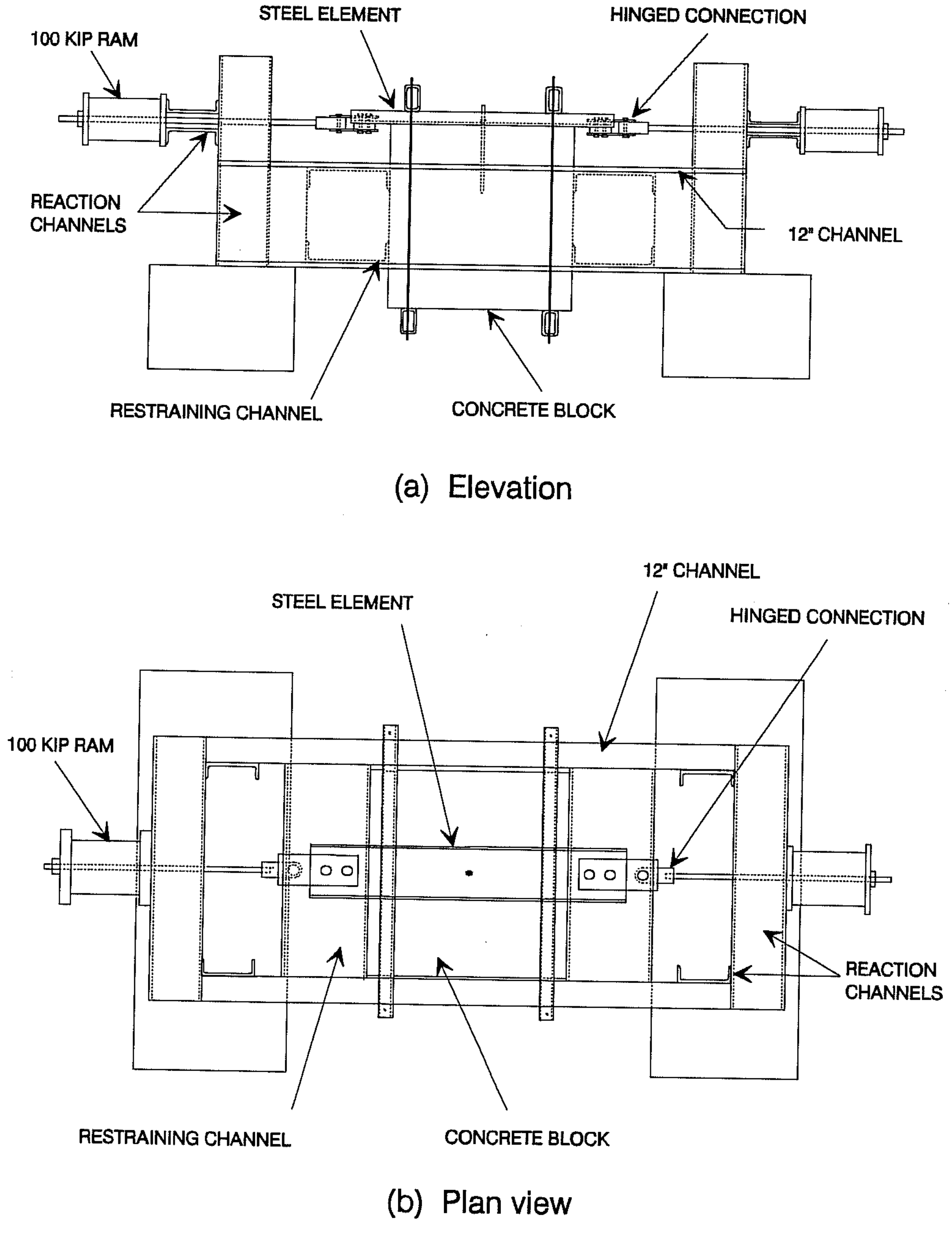


Figure 3.27 Plan view and elevation of test frame

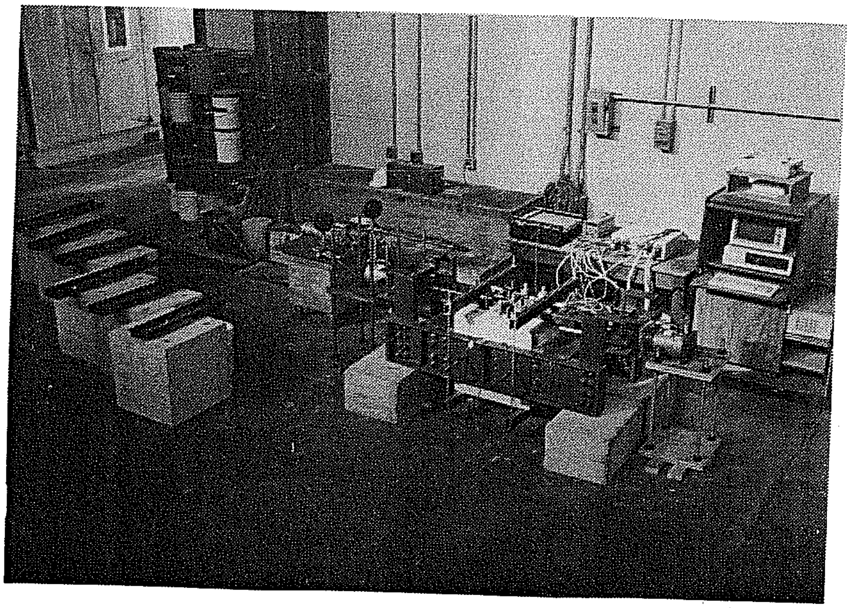


Figure 3.28 Overall view of the test set up with specimens ready for testing

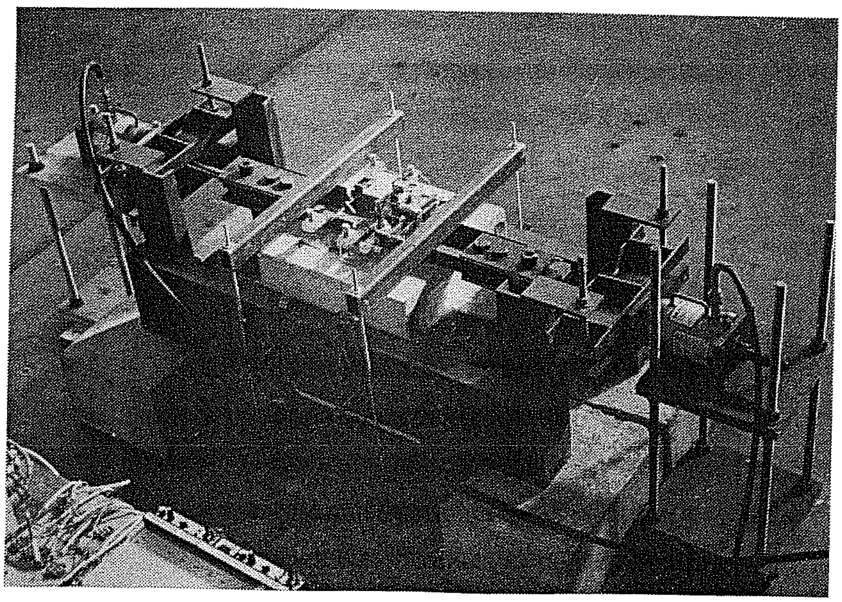


Figure 3.29 Test frame for single-anchor steel-to-concrete connections

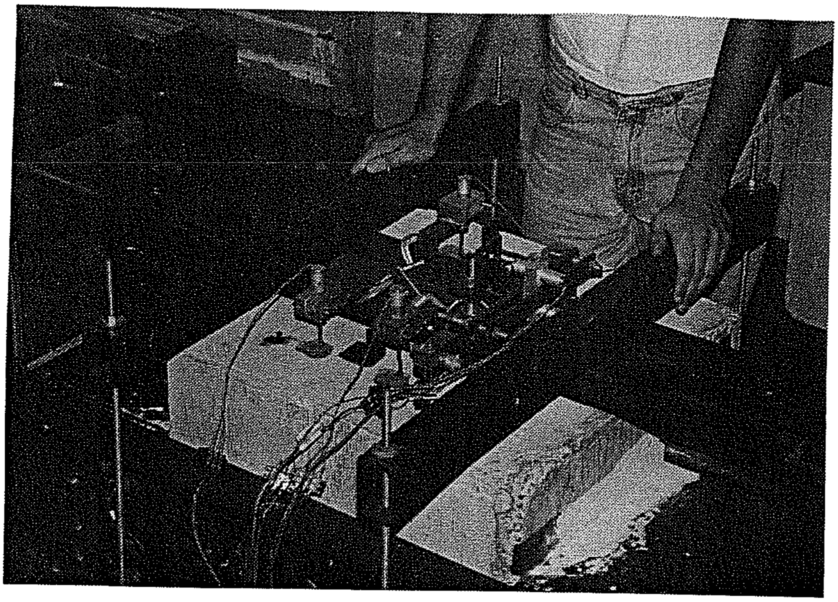


Figure 3.30 Close up view of specimen ready for testing with instrumentation in place

to minimize out-of-plane bending. Furthermore, two pairs of tubes attached to the top and bottom of the specimen prevented excessive uplift of the channel with respect to the concrete surface at all load levels.

3.8. Testing Procedures

3.8.1. Preparation for testing. Figures 3.28 through 3.30 show prepared specimens ready for testing. The restraining girders were pushed against the block after it was set in place. To avoid large concentrations of compressive stress and to prevent movement of the block during testing, voids between the girders and the specimen were filled with hydrastone prior to tightening the bolts.

3.8.2. Testing. For all tests, loading was controlled using a hand pump and a pressure dial gauge. Sufficient data were collected in monotonic tests to get accurate values of loads at first slip and to describe non-linear response up to failure. All specimens in cyclic tests were subjected to a total of eleven load cycles in the four different stages described in Section 4.2.

3.9. Instrumentation and Data Acquisition

3.9.1. Measurement of displacements and deflections. All displacements and deformations were measured using 2-in. linear potentiometers. The location of transducers was the same in all tests and is illustrated in Figs. 3.31 and 3.32. Potentiometers No.1 and No.2 measured interface slip, while transducers No.3 and No.4 were used to calculate bolt rotation. Four other potentiometers were placed on the steel element normal to the interface plane to detect uplift of the channel.

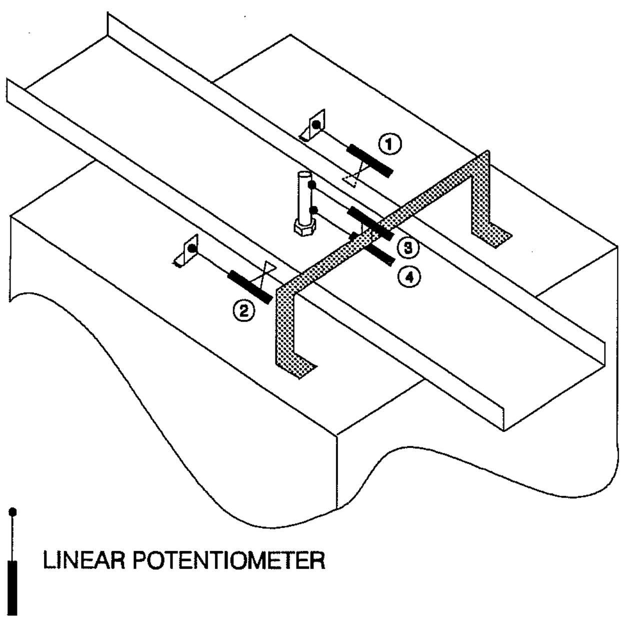


Figure 3.31 Schematic of linear potentiometer layout

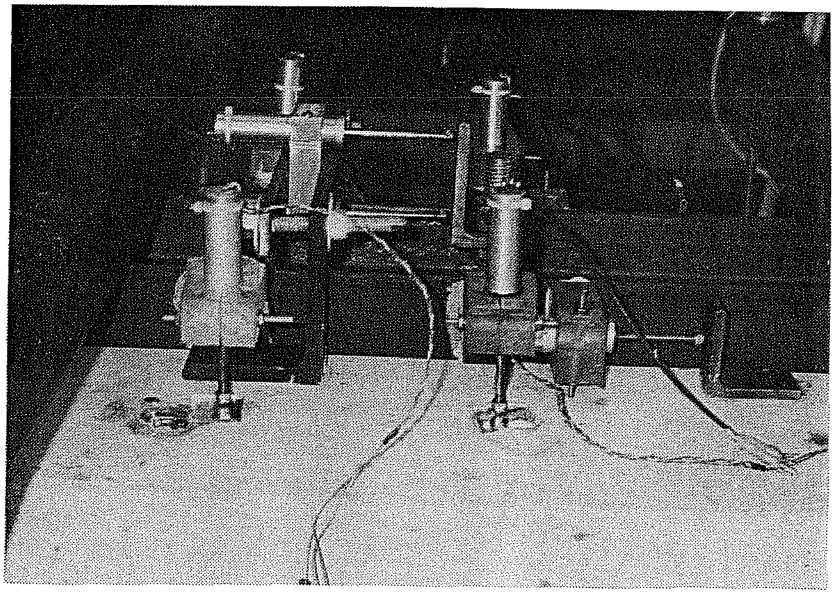


Figure 3.32 Displacement transducers measuring uplift, interface slip, and bolt rotation

3.9.2 Pressure transducers. Load applied to the specimen was monitored using pressure transducers connected in the hydraulic lines leading to each of the two 100 kip rams. Both pressure transducers were calibrated before use with a static hydraulic pressure testing machine.

3.9.3. Data acquisition. The applied load-slip relationship was continuously monitored using an X-Y plotter and was recorded using a computerized data acquisition system. Load application was frequently interrupted to collect data. The number of readings taken was typically above 30 for a monotonic test, and over 140 for a cyclic test. The instrumentation channels were read and a hard copy of the readings was simultaneously obtained in each scanning operation. Measurements were converted to engineering units, stored in microcomputer discs, and reduced and plotted using spreadsheet programs.

CHAPTER 4
PRESENTATION OF EXPERIMENTAL RESULTS

4.1 Introduction

The results of the monotonic and cyclic steel-to-concrete connection tests are presented in this chapter. Observations from the tests and comparisons between selected specimen responses are made. Data are presented in graphical and tabular form. Load-slip results of monotonic tests are organized according to the type of filler material used. Cyclic tests are presented using both complete load-slip plots and envelopes of load-slip response. Anchor bolt deformation data are discussed at the end of the chapter.

4.2 Load History

Connections tested monotonically were loaded in 0.5 kip increments until first slip occurred. Following initial slip, loading was interrupted to collect data whenever significant changes in stiffness occurred.

Specimens tested cyclically were subjected to eleven load cycles characterized by the following stages (See Fig. 4.1):

1) Elastic stage. Two cycles to a maximum load of $\pm$ 5 kips or until permanent slip occurred,

2) Stage A. Three cycles to a maximum interface slip of 0.01 in. in each direction,

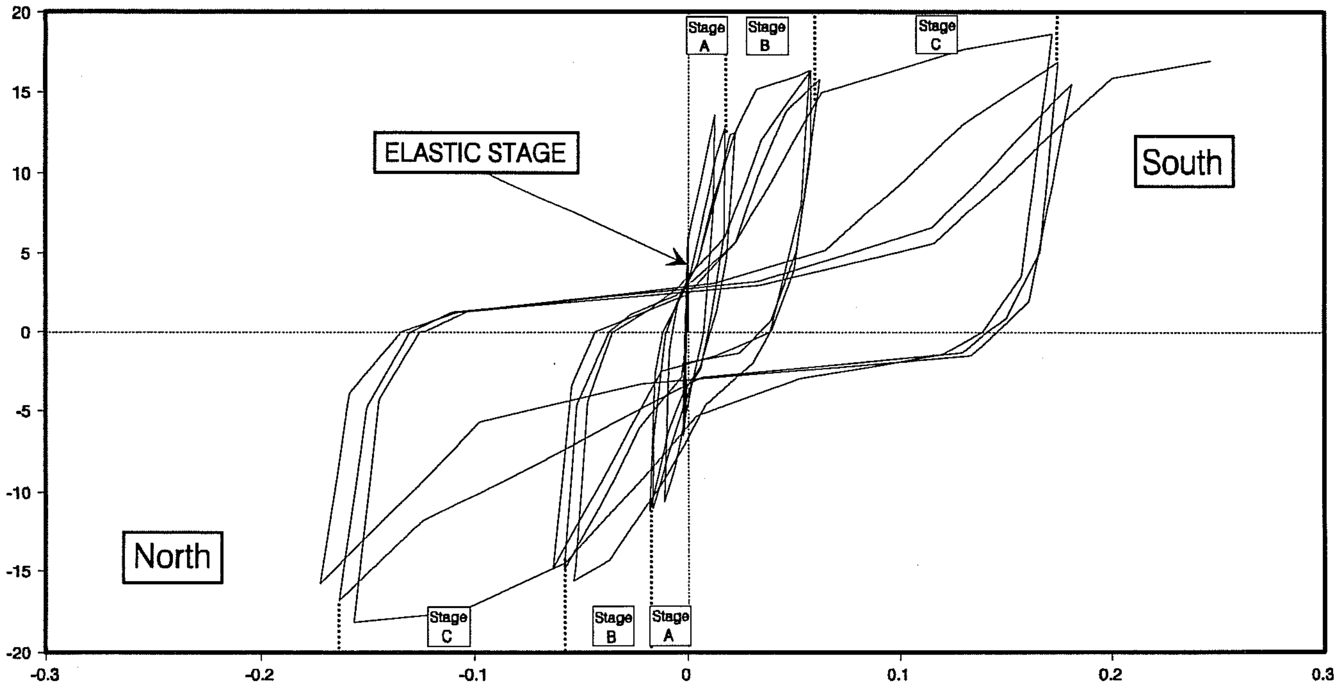


Figure 4.1 Load stages performed in typical cyclic test

3) Stage B. Three cycles to a maximum interface slip of 0.05 in. in each direction,

4) Stage C. Three cycles to an interface slip of 0.15 in., and

5) Failure stage. Monotonic loading until failure of the connection occurred.

Test progress was monitored using a continuous plot of the load versus interface slip response. The applied load dropped during data recording, which resulted in slight discrepancies between peak loads on continuous plots and those captured by the data acquisition system. Slight deviations from the displacement limits occurred in some tests due to creep under sustained load while scanning, partial loss of instrumentation, and small in plane rotations of the steel channel.

Interface slip was measured by linear potentiometers one and two shown in Fig. 3.31. Slight discrepancies in measurements by these two transducers were observed due to rotations of the steel section in the horizontal plane. Data obtained from both devices were averaged to compensate for this difference.

The average number of points recorded was approximately 30 for monotonic tests, and 140 for cyclic tests. Recorded data were manipulated using spreadsheet programs. Furthermore, a portion of the response for some specimens was truncated to facilitate comparisons of results plotted with a common scale.

4.3 Connection Capacities

In Tables 4.1 and 4.2 the maximum load achieved for an interface slip of 0.01 in. or less, the load at 0.05 in., and the peak strength of each test are presented. Cells in

Table 4.1 Load-response data-monotonic tests

Test Identification	Maximum Load Resisted for sip $\leq$ 0.01 in. (kips)	Load Reisited @ 0.05 in. (kips)	Peak Strength (kips)
MN3t	6.2	6.1	17.8
MG3t	5.8	13.3	19.9
MG3t-SS	8.9	13.6	18.8
MG3t-th	7.0	13.5	15.9
MG7t-SS	7.5	13.0	19.6
MG7T-SS	8.6	12.8	21.1
Me3h	4.1	13.7	19.7
Me3t	12.7	18.4	19.5
2-Me3t	14.3	16.0	18.8
Me3t-S	16.0	17.3	20.4
ME3t-IB	6.1	14.9	20.4
Me3t-IS	10.9	14.8	20.3
Me3t-IF	6.0	15.3	20.0
Me3T	14.0	17.1	18.1
Me7h	6.3	14.2	21.1
Me7t	8.3	15.2	21.3
Me7T	23.9	20.8	22.1
ME3h	8.0	14.8	19.2
2-ME3h	17.8	-	20.3
ME3h-SS	32.1	-	22.6
ME3t-SS	44.6	-	20.8

the second column in Tables 4.1 and 4.2 contain the maximum value of load resisted before significant slip occurred. For tests that did not exhibit a distinct point of first relative slip (gradual transition from pre-slip to post-slip behavior), first slip was taken as 0.01 in. The loads reached at an interface slip of 0.01 in. provide an indication of the friction or adhesion capacity of each specimen. An examination of the difference between loads at 0.01 in. and 0.05 in. indicates post slip stiffness.

Table 4.2 Load-response data-cyclic tests

Test Identification	Maximum Load Resisted for slip ≤ 0.01in. (kips)	Load Reisited @ 0.05 in. (kips)	Peak Strength (kips)
CG3t-SS	7.7	12.40	17.2
CG3T-SS	7.8	12.4	15.8
2-CG3T-SS	7.4	12.4	15.9
CG7t-SS	6.7	11.4	18.5
CG7T-SS	7.4	12.5	18.7
CN3h-WN	3.4	9.8	17.0
Ce3t	13.1	16.0	18.6
Ce3T	13.3	16.0	16.0
Ce3t-IB (South)	11.2	10.5	19.2
Ce3t-IB (North)	7.7	12.8	-
Ce7t	14.1	13.6	20.4
Ce3t-SP	13.4	13.7	17.4
Ce3T-SP	17.6	17.0	16.0
Ce7t-SP	6.2	8.3	-
2-Ce7t-SP	11.2	10.3	18.8
Ce7T-SP	12.2	15.4	18.7

The resistance at 0.05 in. slip for tests with interface epoxy is not included in Table 4.1 for reasons explained in Sub-section 4.4.3.2. The peak load of specimen Ce7T-SP was not obtained because the test was not carried out to failure. Loads resisted by specimen Ce3t-IB in both loading directions (South and North) are reported in Table 4.2 since behavior in each direction was substantially different.

4.4 Load Versus Interface Slip-Monotonic Tests

Twenty one specimens were tested monotonically. Load versus interface slip plots for this series of tests are presented in Sub-sections 4.4.1 through 4.4.3. All specimens exhibited very high stiffness until first slip occurred. The maximum load reached before first slip was controlled by friction, adhesion, or a combination of both, and hence varied depending on the specific details of each test. The transition from first slip to bearing of the anchor bolt against the steel section varied as well.

4.4.1 Test without interface materials. A plain steel-to-concrete connection was tested to establish a baseline to which specimens with epoxy or non-shrink grout could be compared. The specimen had a hole with 3/16 in. clearance and was clamped with a force of 12 kips.

The results of this test are plotted in Fig. 4.2. This connection reached a load of 6.2 kips before first slip. After subsequent load application, the steel element slipped on the concrete surface with no increase in load ("friction plateau") until it displaced sufficiently to bear against the bolt. At this point, stiffness increased considerably, remaining fairly constant until the anchor began to deform plastically.

4.4.2 Tests with non-shrink grout. Tests with grout in the annulus and interface revealed a much greater stiffness after first slip than the plain connection, as shown in Fig. 4.3. The cement grout was confined by the nut and washer and effectively

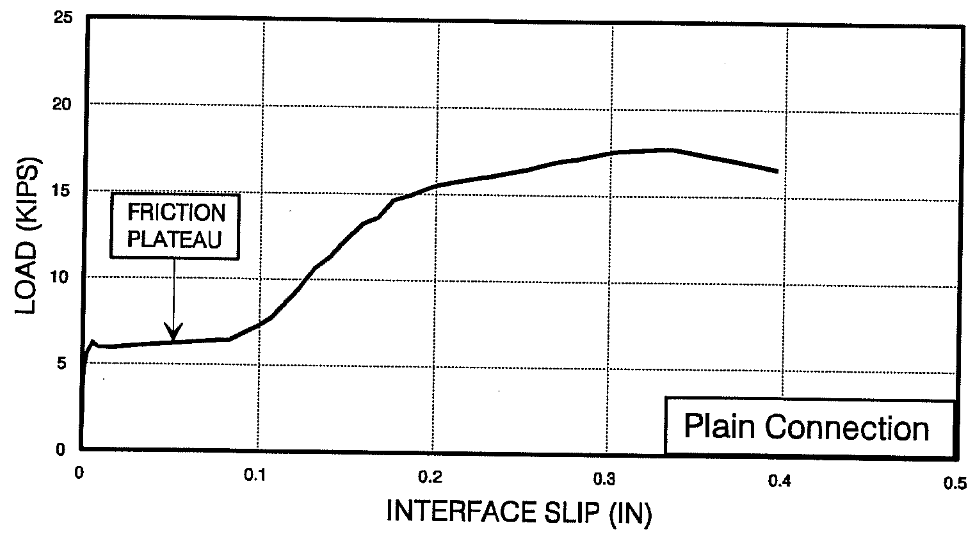


Figure 4.2 Load-slip plot, test MN3t

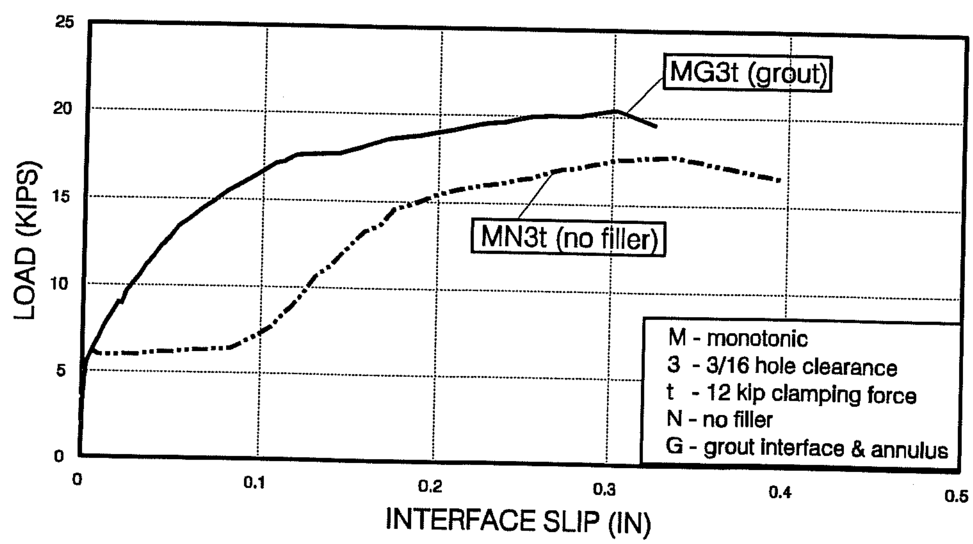


Figure 4.3 Effect of grout in the annulus and interface

filled the annulus. Thus the bolt was engaged at earlier stages of loading and distributed bearing stresses on the steel element more uniformly. In addition, test MG3t had a higher capacity than test MN3t. As others have indicated [45], higher capacity is partly due to increased slip resistance near failure. Figure 4.4 illustrates how bending of the connector assembly at large slip levels results in significant clamping forces applied by the nut and washer on one side of the connector. Abrasions due to frictional wear of the steel section against the base surface were observed in most tests (See Fig. 4.5 and 4.6).

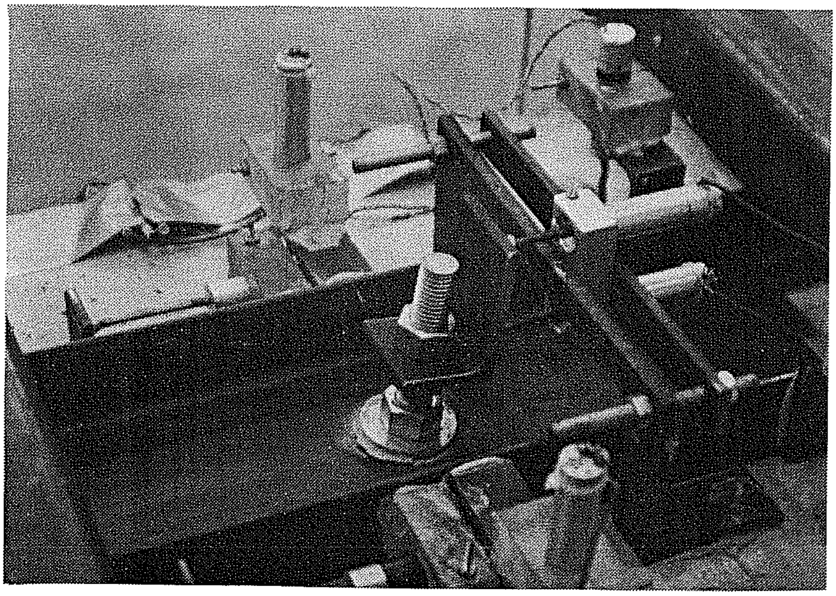


Figure 4.4 Anchor bolt rotation at large relative slip of the steel element

The effect of sandblasting the steel element on load at first slip can be seen in Fig. 4.7. Load at first slip increased by 53 % as a result of surface roughening for tests with otherwise similar details. Surface roughening did not appear to significantly influence post-slip connection behavior. Nevertheless, the load in test MG3t-SS dropped by 1.3 kips after frictional capacity was exceeded. This sudden decrease in load is likely the

Figure 4.5 Frictional wear on grout surface

Figure 4.6 Surface abrasion on steel section

result of non-uniform cement grout filling the annulus. One possibility is that the annulus was not completely filled with grout.

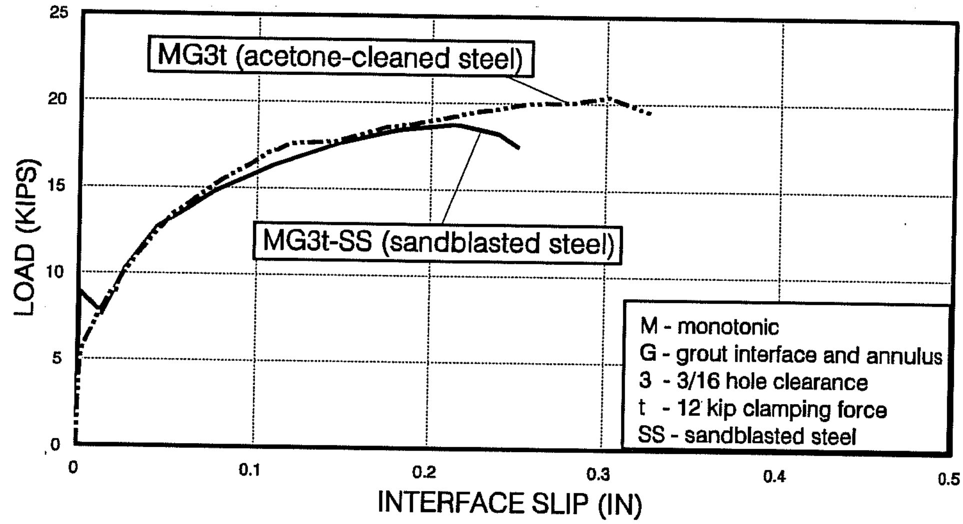


Figure 4.7 Effect of surface roughening by sandblasting (tests with grout in the annulus and interface)

The effect of clamping force on two tests with cement grout is illustrated in Fig. 4.8. For these two tests, first-slip capacity increased proportionally to the difference in applied clamping. Load at first slip was approximately equal to 50% of the initially applied clamping force in both cases. As shown in the figure, clamping level did not have an important influence on post-slip connection behavior.

Figure 4.9 shows a comparison of two tests with the same clamping force but different hole oversize. The post-slip behavior of both specimens was similar up to an interface displacement of 0.05 in. However, the connection with 3/16 in. hole clearance reached its peak capacity earlier than the connection with 7/16 in. clearance.

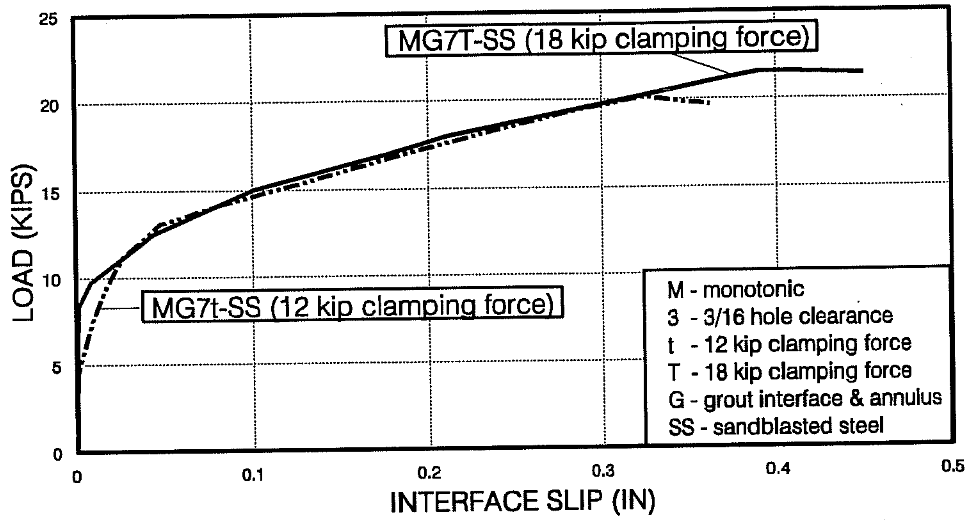


Figure 4.8 Effect of increased clamping force (tests with grout in the annulus and interface)

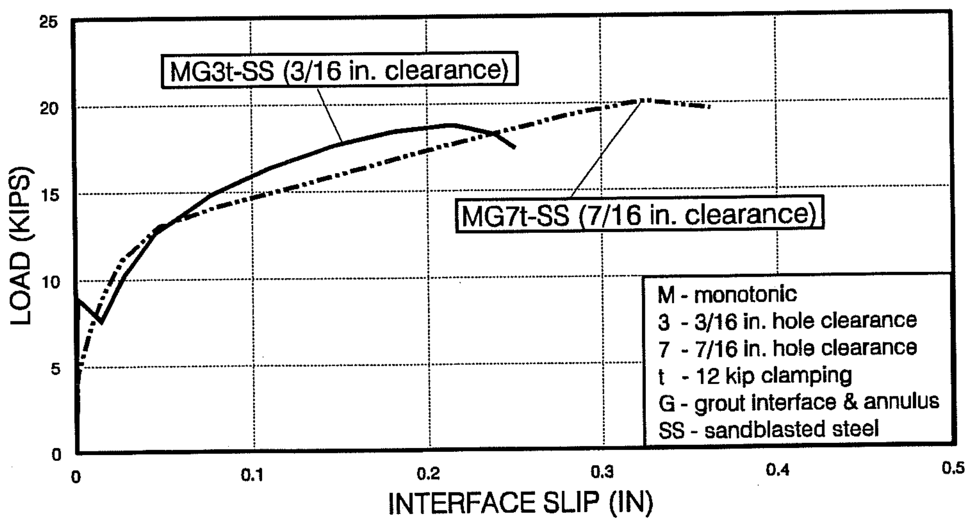


Figure 4.9 Effect of hole clearance (tests with grout in the annulus and interface)

The behavior of a connection with a 1/2 in. thick interface is presented in Fig. 4.10. After initial slip, test MG3t-th behaved similar to specimen MG3t up to a relative displacement of .08 in. At this point, the grout interface cracked, as shown in Fig. 4.11 and 4.12, and the load dropped. With the interface cracked, the anchor bolt was able to deform in bending which increased the ductility of the connection. Because the bolt was subjected to a combination of shear and significantly more bending, the ultimate capacity decreased. The specimen failed at a load of 15.9 kips and interface slip of 0.74 in.

4.4.3 Tests with epoxy

4.4.3.1 Tests with epoxy-filled annulus. For this series of tests, epoxy was applied to the base of the anchor bolt and surrounding concrete as explained in Sub-section 3.6.5.2. Substantial adhesion was observed in some tests. Most specimens had a roughly circular bonded surface area of 3 to 4 inches in diameter (See Figs. 4.18 through 4.20).

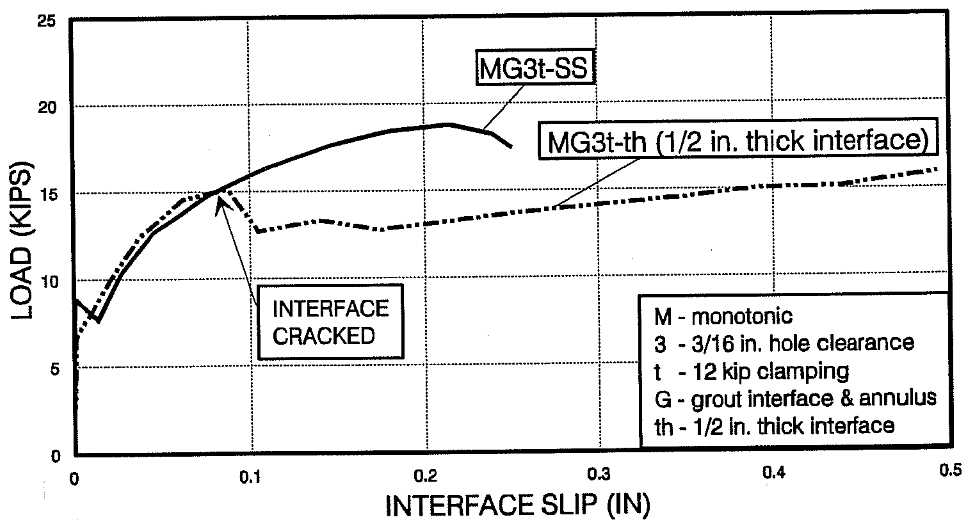


Figure 4.10 Effect of interface thickness (tests with grout in the annulus and interface)

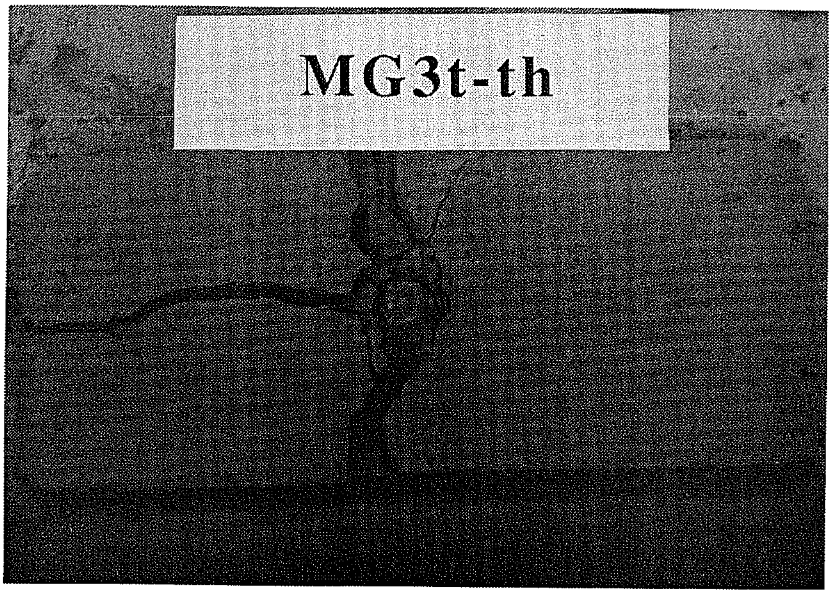


Figure 4.11 Cracking of grout interface in test MG3t-th

Figure 4.12 Anchor bolt showing effect of permanent flexural deformations above concrete surface

The post-slip response of specimens with epoxy in the annulus was satisfactory. As shown in Fig. 4.13, hole oversize did not significantly affect connection stiffness. Furthermore, hand tightened connections with epoxy behaved similar to torqued connections with grout after first slip (See Fig. 4.14). However, resistance to first slip was low for hand-tightened specimens.

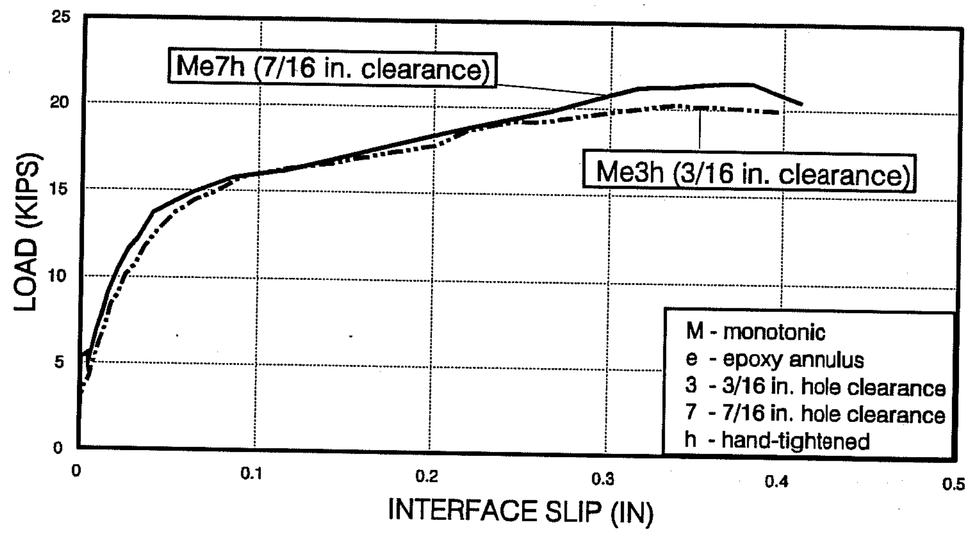


Figure 4.13 Effect of hole clearance (hand-tightened specimens with epoxy-filled annulus)

The effect of increased clamping force on connections with a hole clearance of 3/16 in. is illustrated in Figs. 4.15 through 4.17. Stiffness was greatly improved by clamping force for specimens with 3/16 in. oversize holes. As shown in Fig. 4.15, tests with a clamping force of 12 kips approached an ideal elasto-plastic type of behavior. The fact that the rod was located on the side of the hole in specimen Me3t-S did not adversely affect the response (See Fig. 4.16). An examination of Fig. 4.17 shows that an additional increase in clamping force to 18 kips did not improve connection performance.

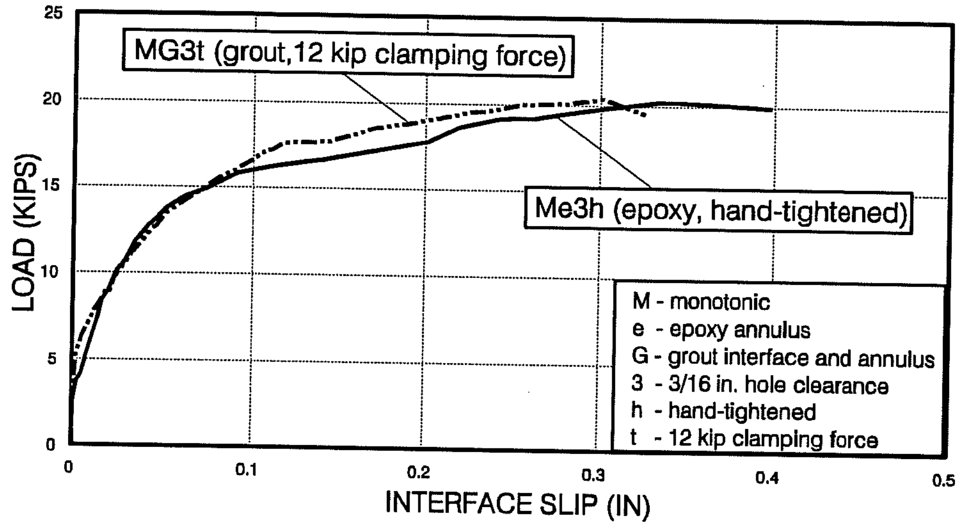


Figure 4.14 Comparison of hand-tightened test with epoxy-filled annulus versus torqued connection with non-shrink grout

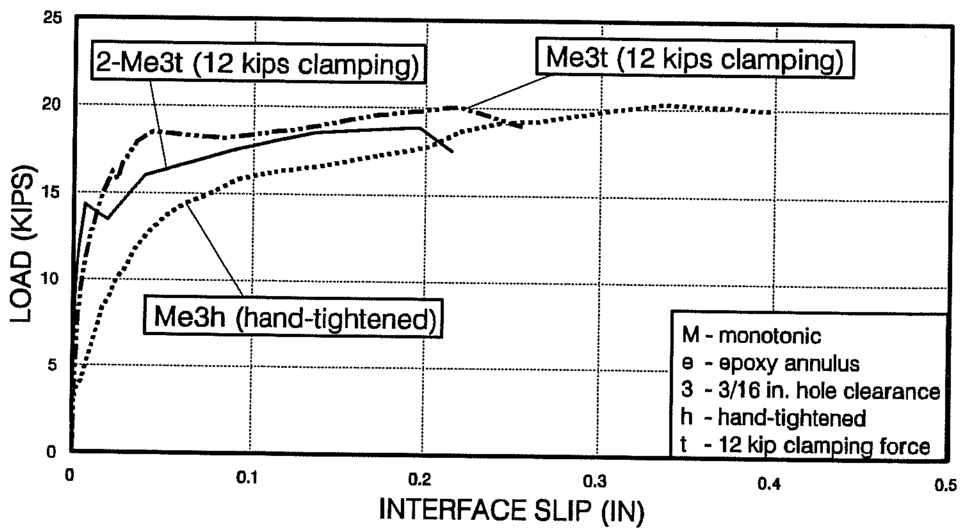


Figure 4.15 Effect of clamping force (specimens with epoxy-filled annulus and 3/16 in. hole clearance)

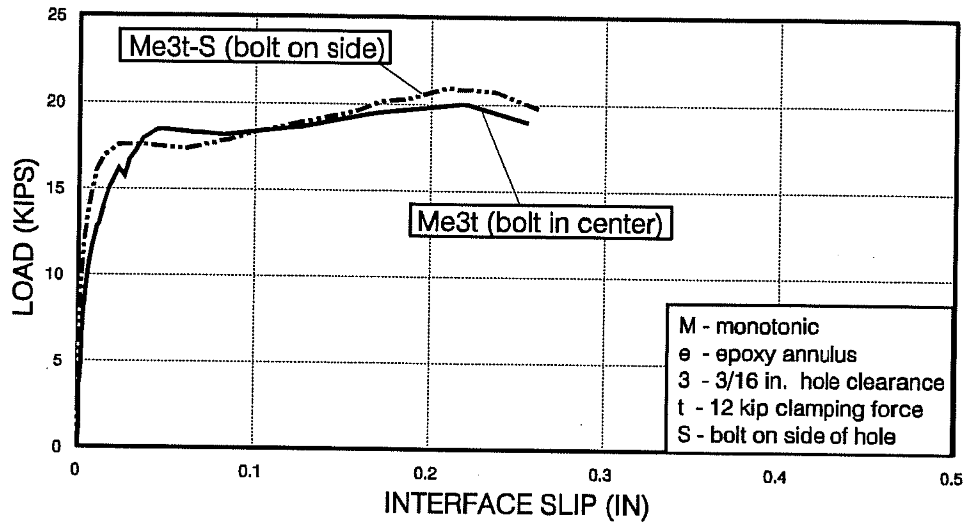


Figure 4.16 Effect of bolt positioned on the side of the hole in the steel element (tests with epoxy-filled annulus)

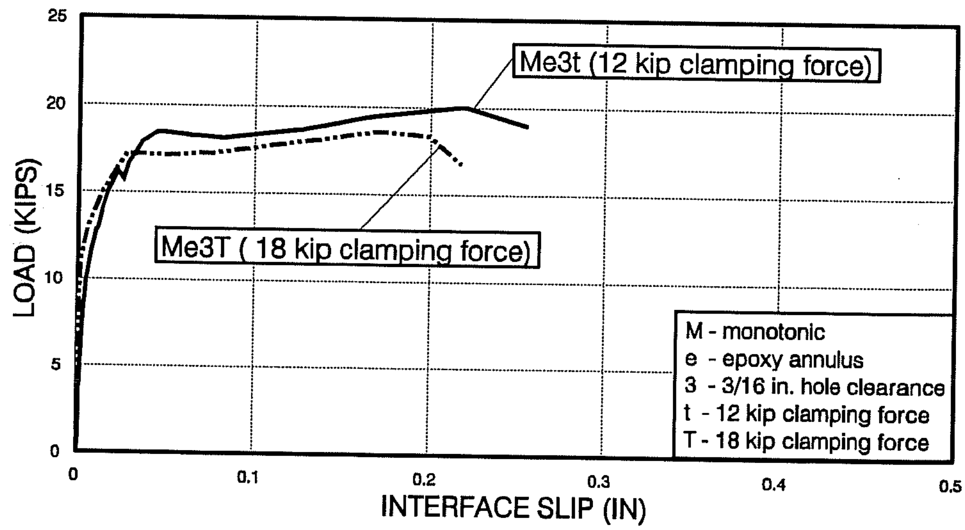


Figure 4.17 Effect of increasing clamping force to 18 kips (tests with epoxy-filled annulus and 3/16 in. hole clearance)

Different modes of failure of the epoxy extruded at the interface caused slight differences in first-slip behavior. For example, as shown in Figs. 4.18 and 4.19, failure of the epoxy at the interface propagated through both the concrete and steel surfaces for tests Me3t and Me3t-S. These two tests showed a smooth transition from elastic to post-slip response. However, epoxy in test 2-Me3t debonded completely from the steel surface (See Fig. 4.20). In this case, the transition from elastic to post slip behavior was characterized by a sudden drop in load and increase in displacement. Results of test 2-Me3t are used in following comparisons because its post-slip strength was less than test Me3t.

Results of tests with 7/16 in. hole clearance and different levels of clamping are shown in Figs. 4.21 and 4.22. The difference in behavior between a hand tightened test and a connection with a clamping force of 12 kips was almost negligible. Clamping force

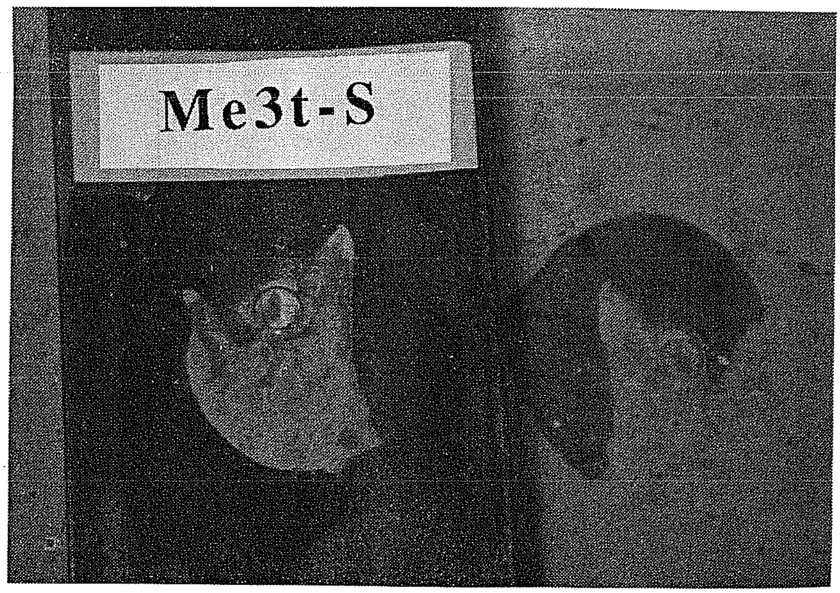


Figure 4.18 Epoxy bonded to both concrete and steel surfaces after failure - Test Me3t-S

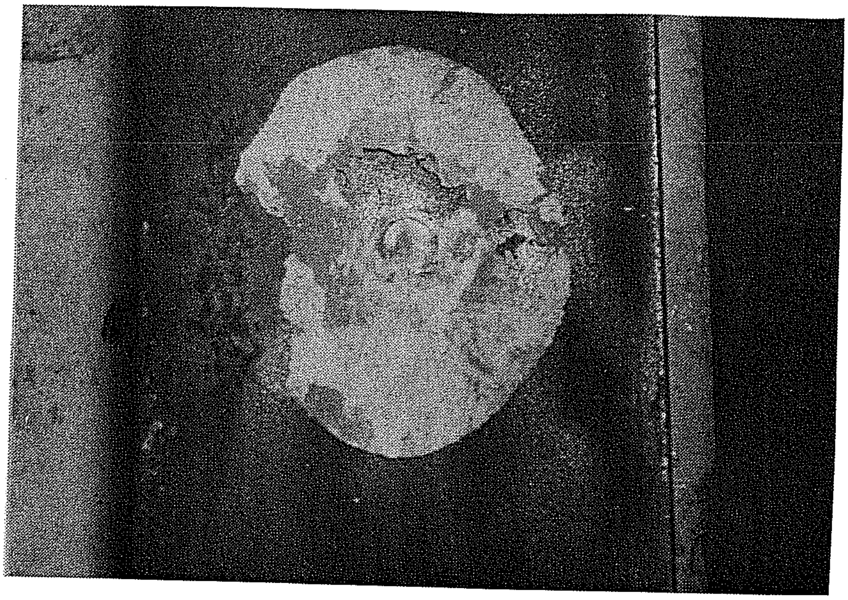


Figure 4.19 Failure of epoxy at both concrete and steel surfaces - Test Me3t

Figure 4.20 Epoxy bonded to concrete surface only - Test 2-Me3t

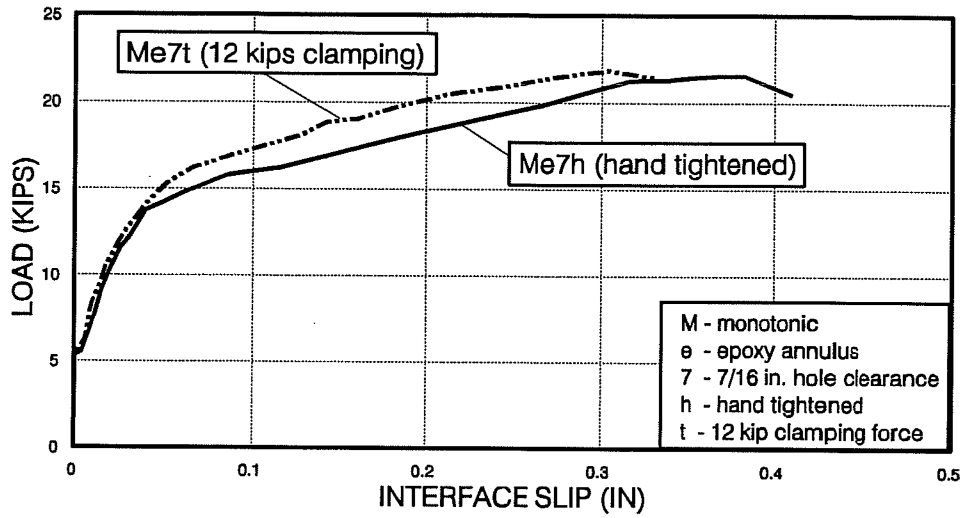


Figure 4.21 Effect of clamping force (tests with epoxy-filled annulus and 7/16 in. hole clearance)

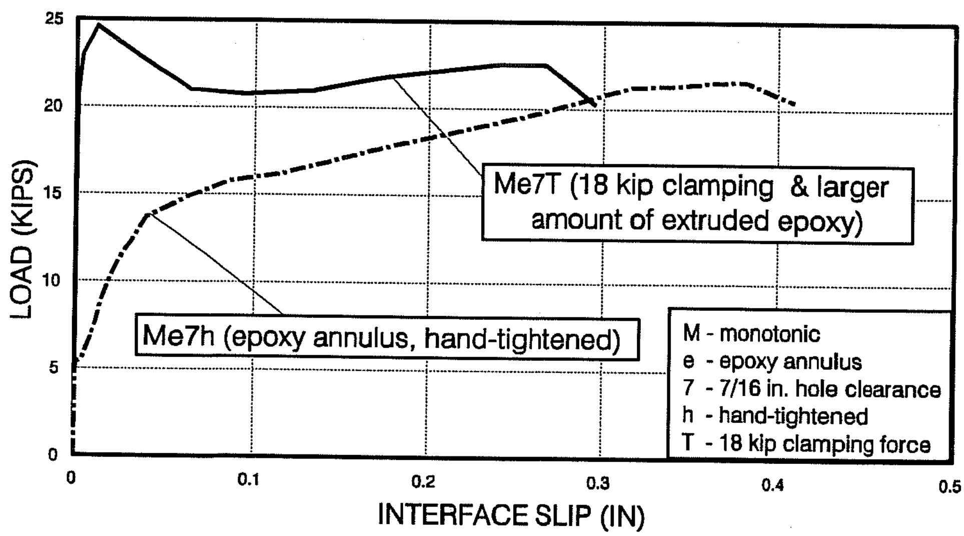


Figure 4.22 Effect of additional clamping force and larger extruded epoxy area (tests with epoxy-filled annulus and 7/16 in. hole clearance)

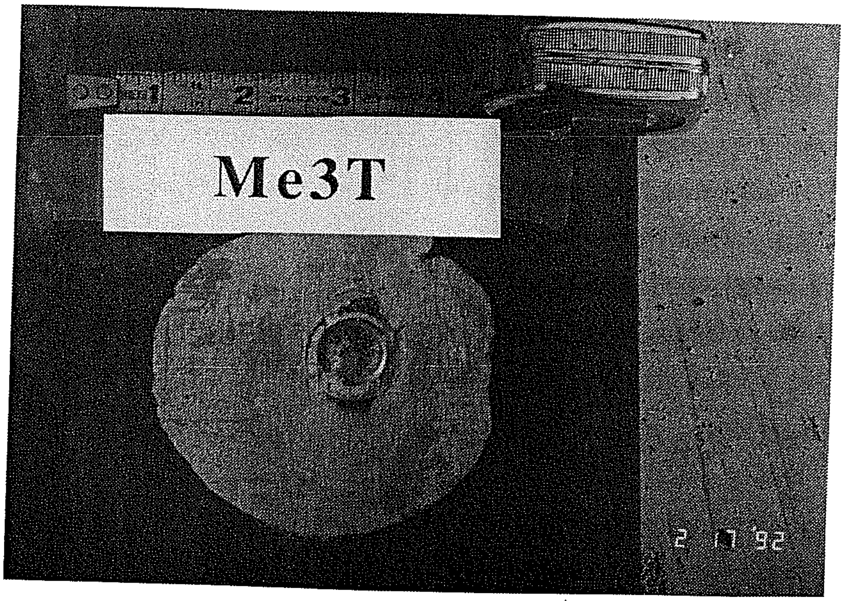


Figure 4.23 Bonded area in test Me3T - approximately 3 in. in diameter

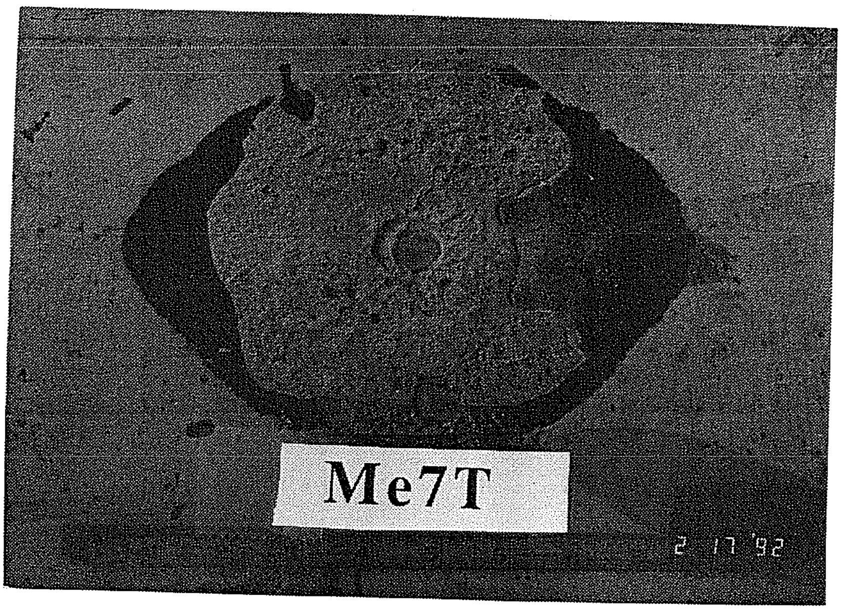


Figure 4.24 Bonded area in test Me7T - approximately a 7 in. by 6 in. elliptical area

slightly improved post-slip response but did not increase load at first slip. Even though higher elastic and ultimate capacities were obtained for test Me7T, the improvement was mainly attributed to the larger area of the bonded interface associated with this specimen. Whereas most specimens had a circular bonded area with a diameter of 3 to 4 inches, test Me7T had an elliptical interface epoxy area of about 7 in. by 6 in. (See Figs. 4.23 and 4.24).

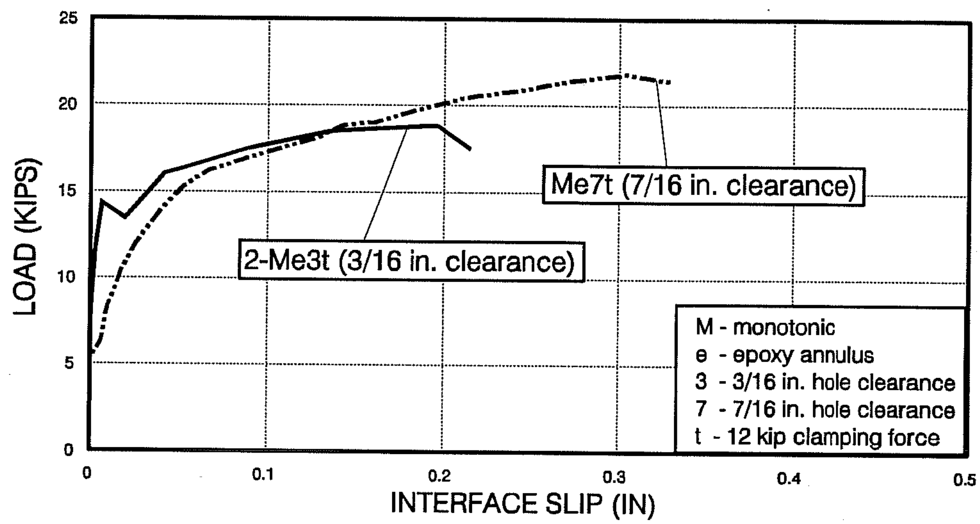


Figure 4.25 Effect of hole clearance (tests with epoxy-filled annulus and 12 kip clamping force)

Figure 4.25 illustrates the effect of hole clearance for a level of clamping of 12 kips. As shown in the plots, increased hole clearance considerably decreased post-slip stiffness thereby increasing relative slip between the steel element and the concrete block. Consequently, specimens with a 3/16 in. hole clearance reached the peak capacity earlier than specimens with a 7/16 in. hole clearance.

Behavior of tests with epoxy in the annulus was altered by initial bolt inclination. Figure 4.26 compares the response of two connectors having an inclination of 5 degrees with a properly installed anchor bolt. The plot of test Me3t-IS is presented separately in

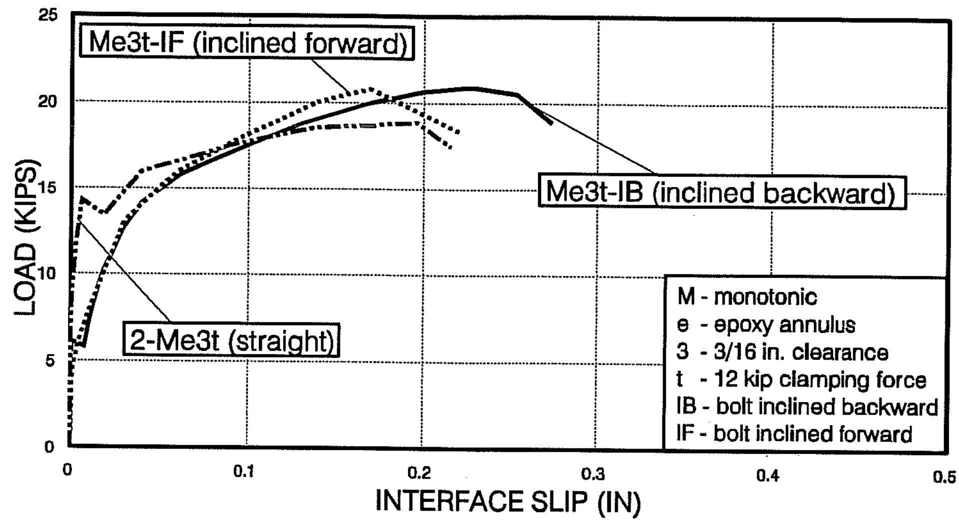


Figure 4.26 Effect of 5 degree bolt inclination parallel to the direction of loading (tests with epoxy-filled annulus and 3/16 in. hole clearance)

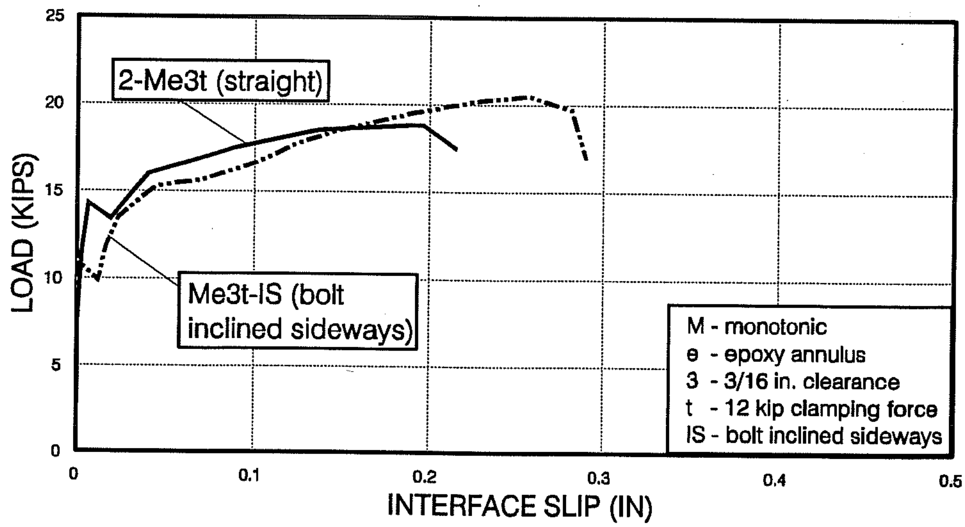


Figure 4.27 Effect of 4 degree bolt inclination perpendicular to the direction of loading (tests with epoxy-filled annulus and 3/16 in hole clearance)

Fig. 4.27 since its initial inclination was equal to 4 degrees. All three tests with inclined anchors demonstrated a lower first-slip capacity than the straight connector. As discussed in Sub-section 2.3.3, initial bolt inclination increases short term connection relaxation due to a greater amount of localized plastic deformations in the fastener components. Post slip behavior was also affected and varied depending on the direction of inclination with respect to the direction of loading (See Fig. 4.28). As the bolt deforms in a connection with the bolt inclined backward (away from the direction of loading), the clamping force provided by the anchor bolt is lost, and as a result, the friction component of resistance is also depleted. Friction is not recovered until the bolt bends enough to clamp down on the loaded side of the steel element. On the other hand, in a connection with the anchor bolt leaning forward, the nut and washer are clamped and remain clamped against the steel element from the onset of loading. Consequently, test Me3t-IF reached peak strength at a smaller displacement than test Me3T-IB.

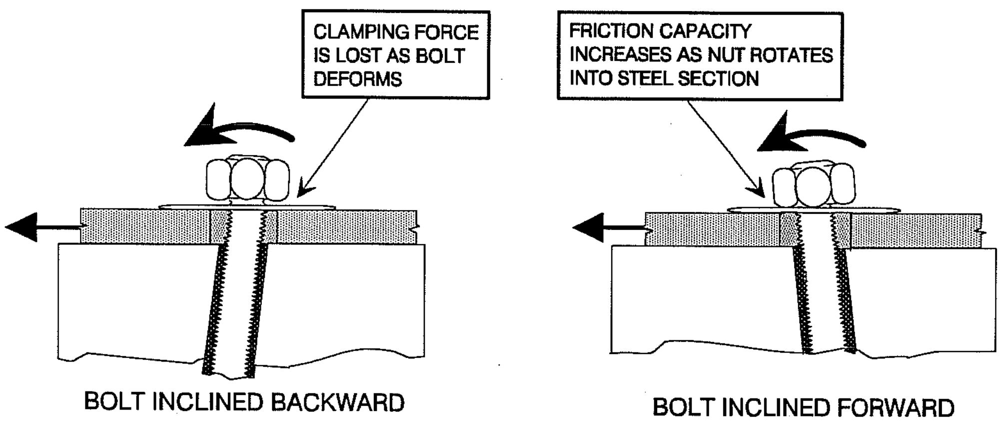


Figure 4.28 Schematic of behavior of inclined anchors

4.4.3.2 Tests with interface epoxy. Tests with interface epoxy exhibited a significant increase in strength before first slip. Results of tests with an epoxy bonded interface of 6 in. by 12 in. are shown in Fig. 4.29. For all three tests, slip did not occur until the elastic strength was in excess of the predicted ultimate bolt shear capacity.

Roughening of the steel element increased the shear capacity of the bonded area by a factor of two. As shown in Figs. 4.31 and 4.32, the epoxy interface completely debonded from the steel surface in test 2-ME3h, whereas interface failure propagated through a portion of the concrete surface in test ME3h-SS.

Clamping force additionally increased strength before first slip for this test series. An examination of Fig. 4.29 and the values in Table 4.1, show that an applied clamping force of 12 kips increased elastic strength by roughly the same amount (12 kips).

The large first-slip capacities obtained in these single-anchor connection tests with interface epoxy are not reliable for design of steel-to-concrete connections. The exact transition from pre-slip to post-slip behavior could not be recorded because failure of the epoxy interface was very sudden. For this reason, the portion of the response curves corresponding to the transition between the data recorded before and after the interface debonded is indicated by dashed lines in Fig. 4.29. In addition, values of resistance at 0.05 in. interface slip for this test series were not included in Table 4.1. As revealed by Fig. 4.29, the load dropped drastically and the steel channel slipped a large amount (from 0.04 in. to as much as 0.21 in.) upon failure of the epoxy interface. This drop in load and increase in relative slip was more pronounced as the capacity of the epoxy interface increased.

Plots after first slip for this set of tests do not reflect observed behavior of previously tested multiple-anchor connections. The abrupt increases in displacement caused by the sudden transfer of load from the bonded interface to the anchor-epoxy

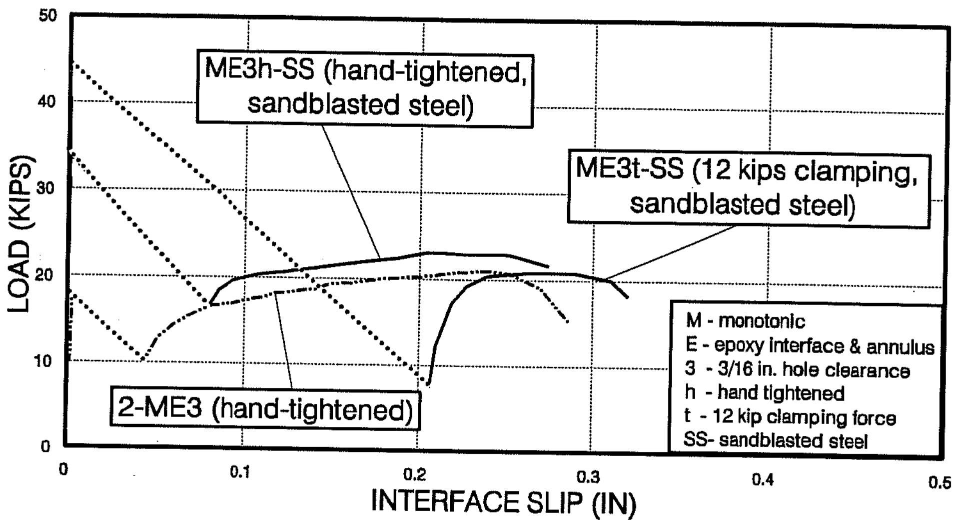


Figure 4.29 Effect of interface epoxy (3/16 in. hole clearance)

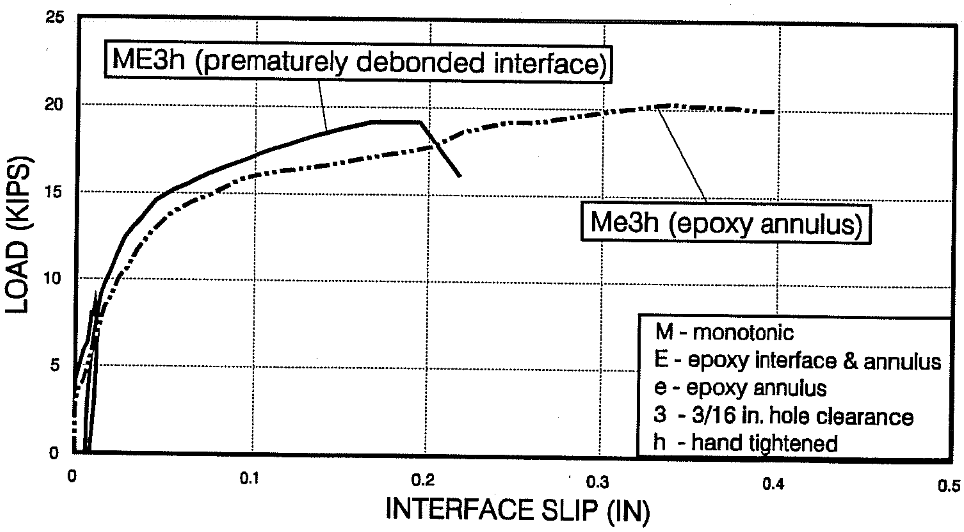


Figure 4.30 Comparison of test with debonded epoxy interface and test with epoxy-filled annulus (hand-tightend tests with 3/16 in. hole clearance)

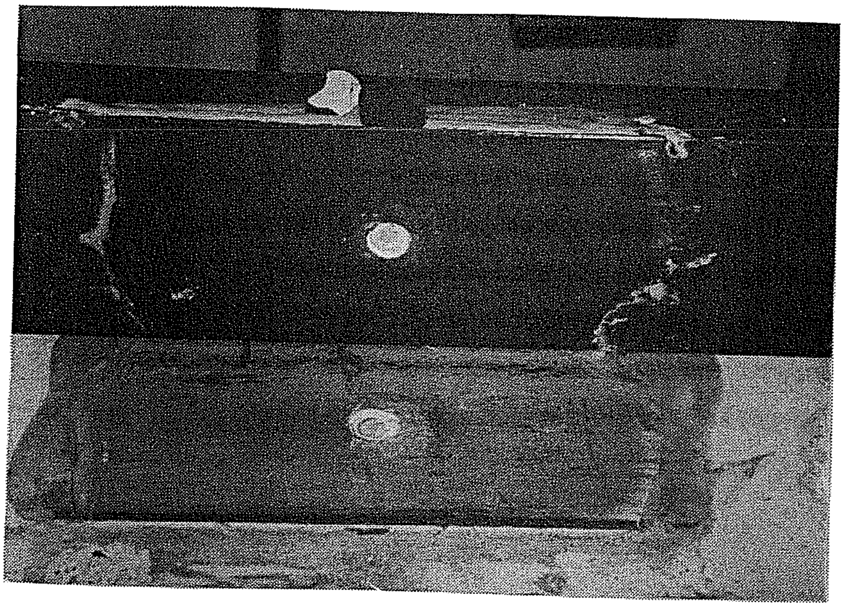


Figure 4.31 Epoxy interface completely debonded from steel section - Test 2-ME3h

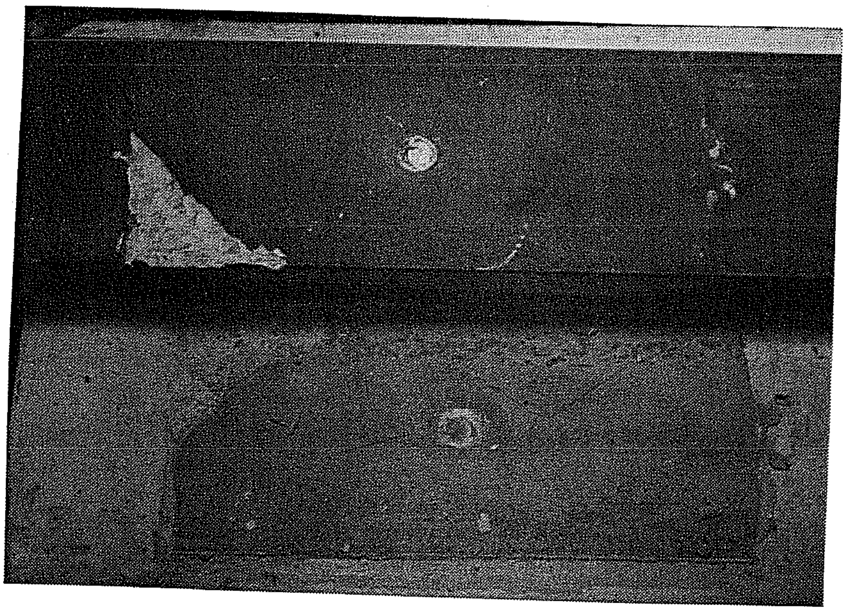


Figure 4.32 Concrete bonded to steel section by epoxy - Test ME3h-SS

filler system, were not observed in previous research by Wiener on multiple-anchor connections where load was distributed among several bolts after first slip [45].

Test ME3h was repeated because the interface debonded at an early stage due to misalignment of the loading devices. Results of test ME3h are compared in Fig. 4.30 with a specimen having epoxy in the annulus only. The only observed difference is that test ME3h reached the peak load at a lower displacement than test Me3h. Interface epoxy may have contributed to slightly increase post slip stiffness. Cracking of the interface epoxy near the anchor bolt (Fig. 4.33) indicates the epoxy may have increased the stiffness of the single-anchor connection.

Figure 4.33 Cracking in tension of interface epoxy - Test ME3h

4.5. Response to Cyclic Loading

4.5.1 Load versus interface slip

4.5.1.1 Standard connections. Nine specimens with standard nuts and washers were loaded cyclically, as described in Section 4.2. Load versus interface slip responses for each of the nine tests are presented in Figs. 4.36 through 4.44. Tests generally showed very high stiffness in the elastic stage. Connection stiffness decreased progressively in other stages under repeated cycling.

General characteristics of connection behavior are illustrated in Fig. 4.34. As the steel element slips over the concrete surface, load is transferred from the steel element to the bolt through the filler material, and, from the bolt to the concrete through the epoxy used to attach the anchor bolt to the concrete member (See Fig. 4.35-a). Degradation observed in load stages A and B (Fig. 4.1) is likely the result of some permanent damage in the epoxy surrounding the anchor bolt in the concrete block and the material in the annulus (local crushing of grout or plastic deformation of the adhesive), perhaps some

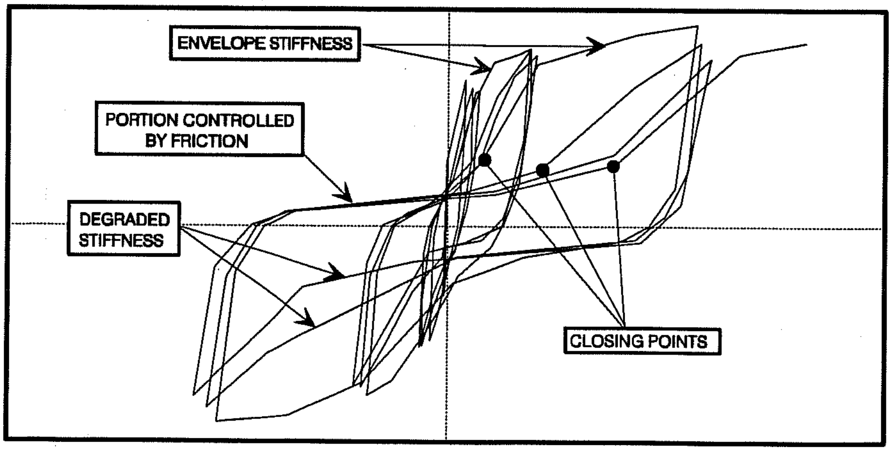
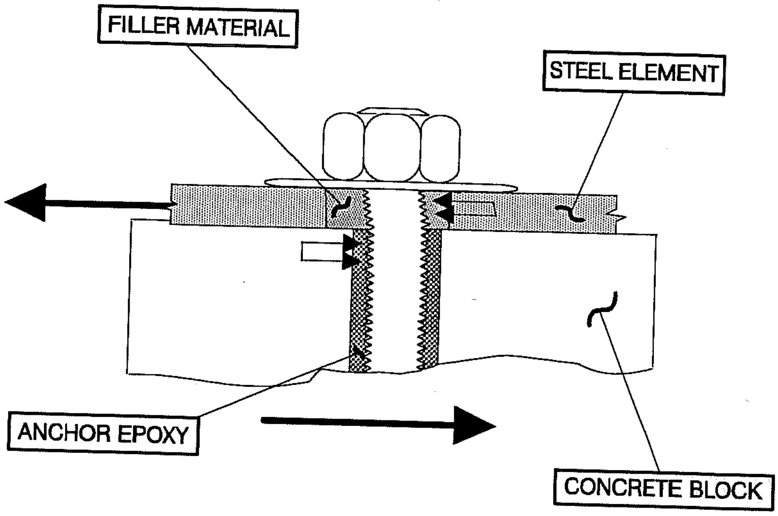
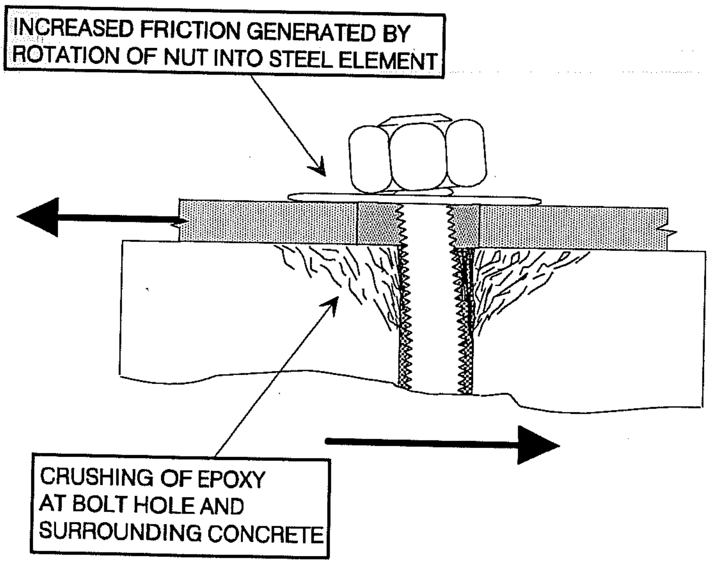


Figure 4.34 Events characterizing response to cyclic loading of steel-to-concrete connections



(a) Load transfer in steel-to-concrete connection



(b) Behavior at large displacements

Figure 4.35 Schematics of behavior of steel-to-concrete connection

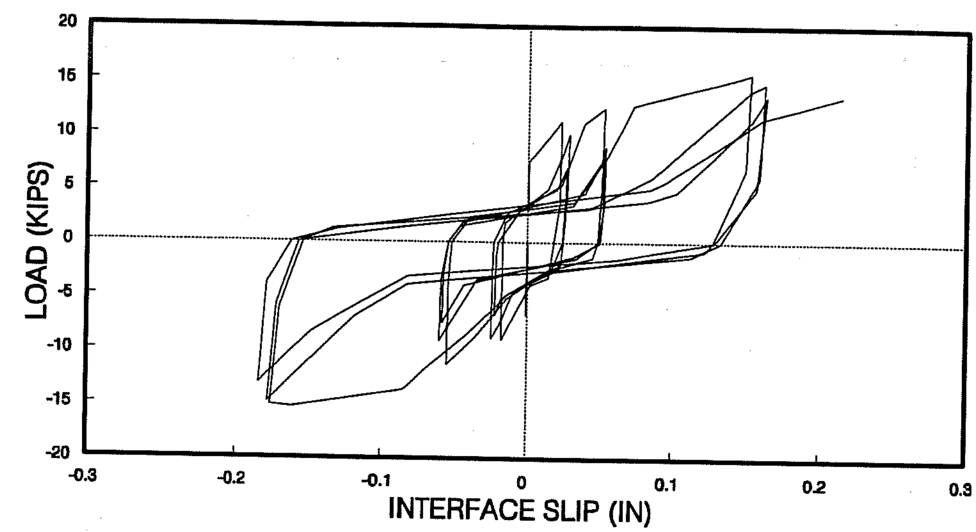


Figure 4.36 Load-slip plot - Test CG3t-SS

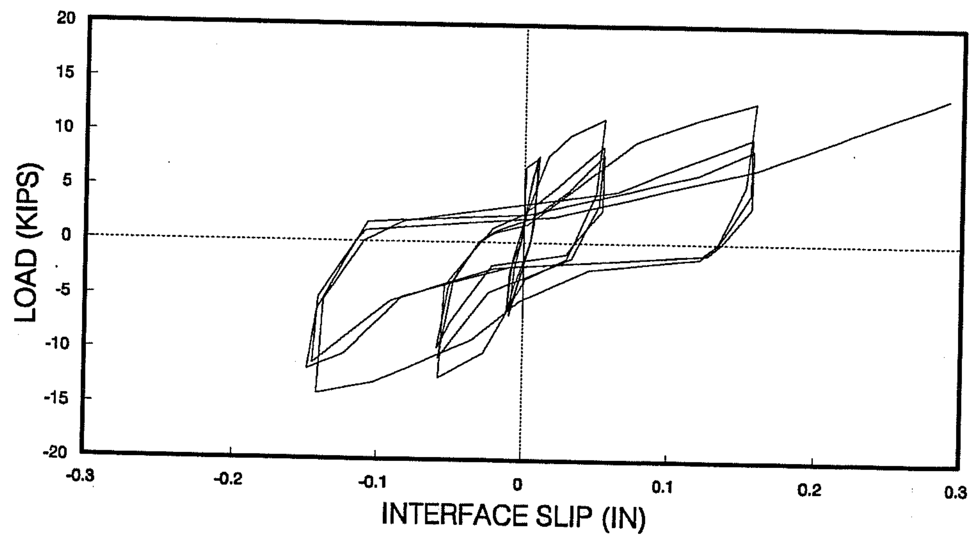


Figure 4.37 Load-slip plot - Test CG7t-SS

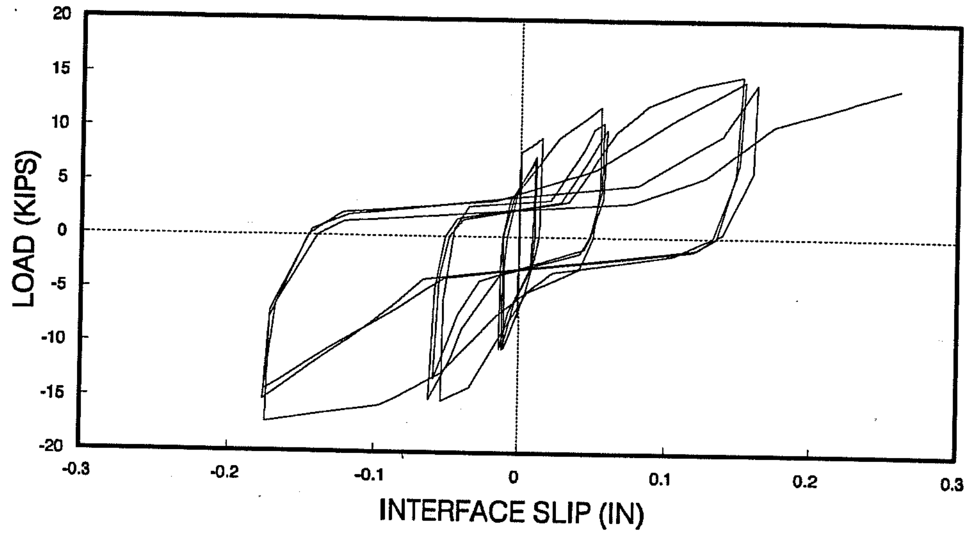


Figure 4.38 Load-slip plot - Test CG3T-SS

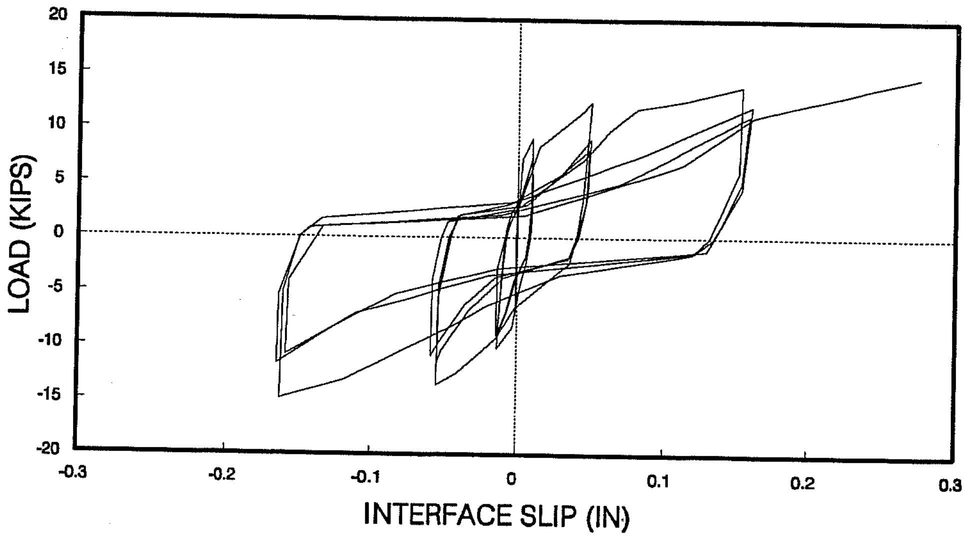


Figure 4.39 Load-slip plot - Test CG7T-SS

crushing of the surrounding concrete. In load stage C, resistance after load reversals is primarily characterized by friction and flexural stiffness of the anchor bolt. At large displacements, most of the original clamping force in connectors with standard washers is lost since the bolt has already deformed plastically. In addition, flexural stiffness of the anchor bolt decreases as concrete surrounding the anchor crushes (See Fig. 4.35-b). Stiffness does not increase until the bolt reaches the closing points denoted in Fig. 4.34, corresponding to bearing against undamaged material.

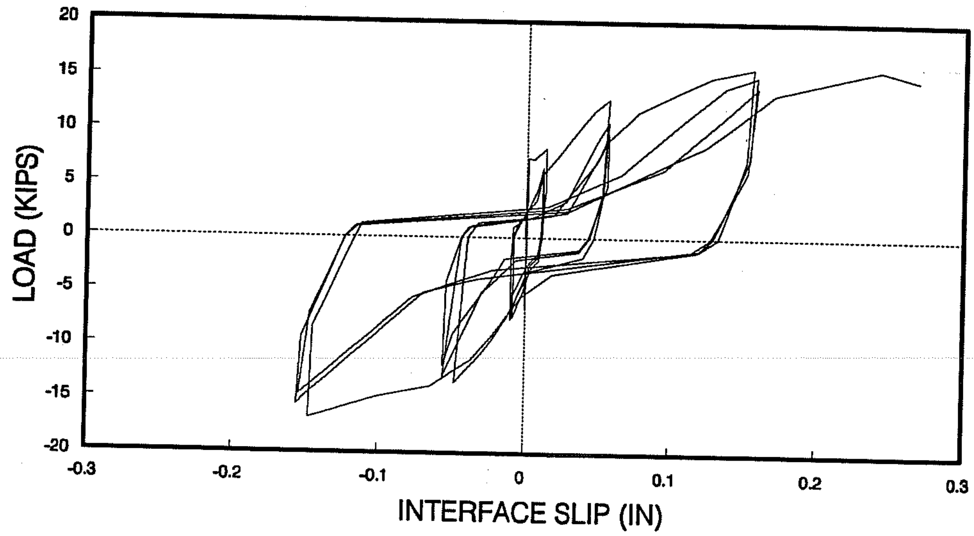


Figure 4.40 Load-slip plot - Test 2-CG3T-SS

Connections incorporating non-shrink grout and epoxy exhibited similar characteristics in their response to cyclic loading. Clamping force and bolt inclination did not influence the rate of connection degradation. However, more "softening" was observed for tests with greater hole clearance. The greater the amount of filler material participating in connection resistance, the higher the stiffness degradation rate with repeated loading.

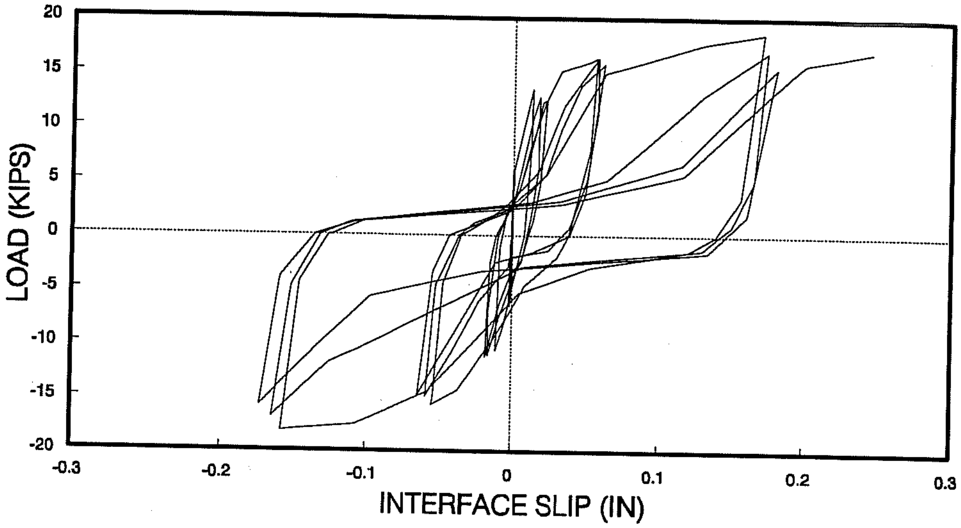


Figure 4.41 Load-slip plot - Test Ce3t

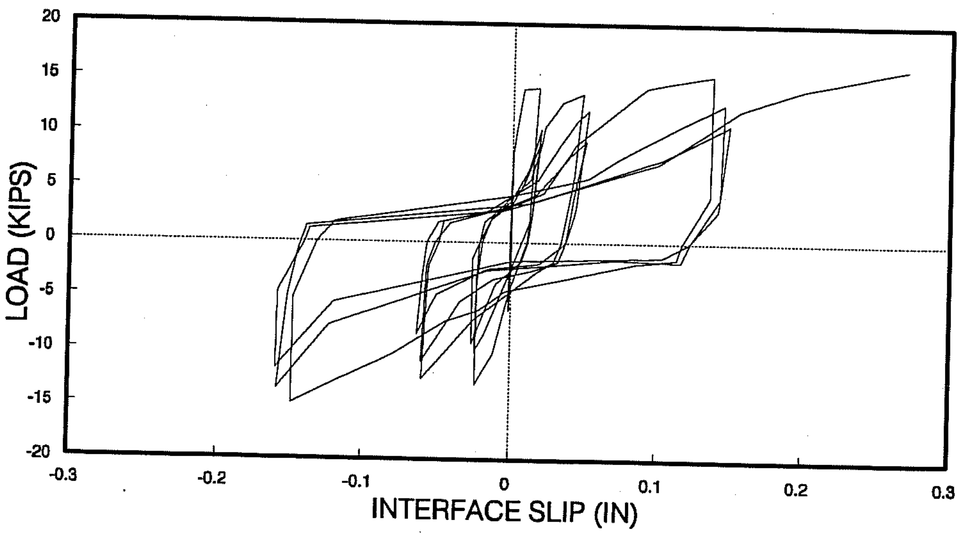


Figure 4.42 Load-slip plot - Test Ce7t

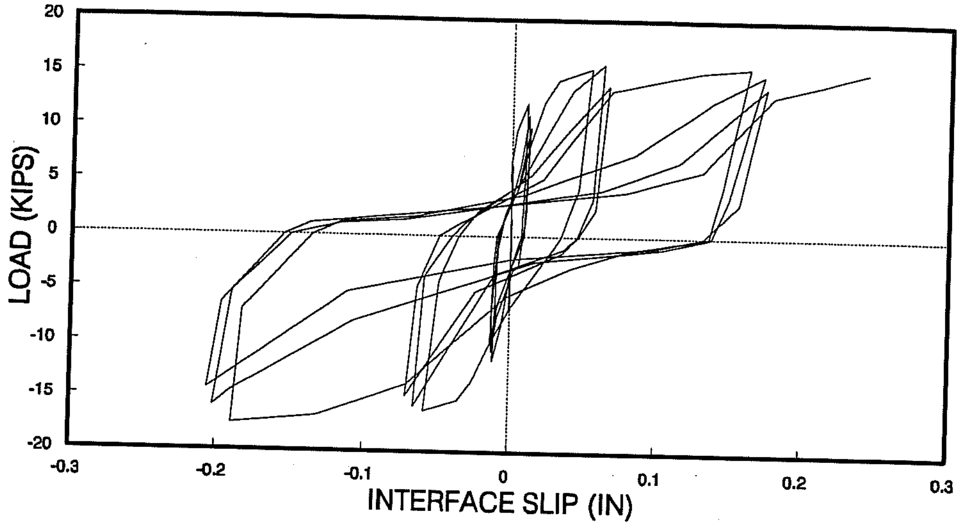


Figure 4.43 Load-slip plot Test Ce3T

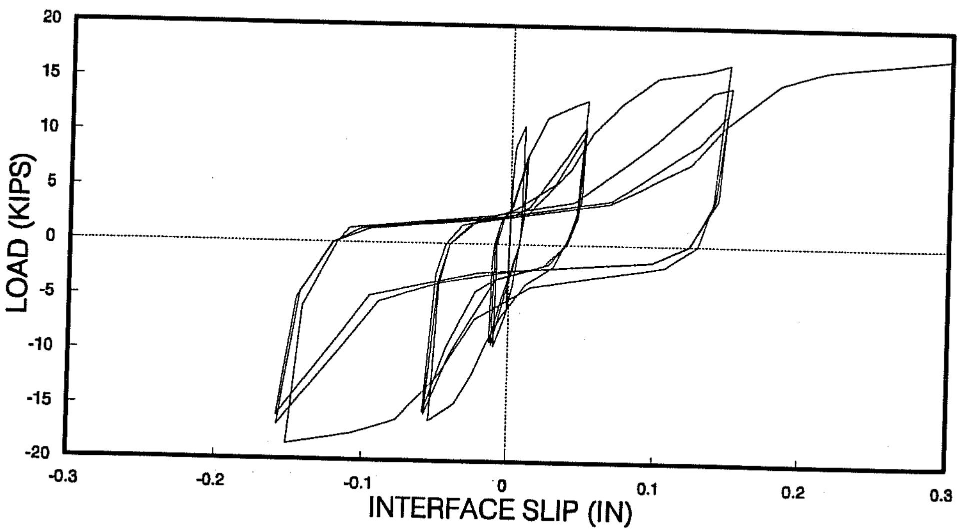


Figure 4.44 Load-slip plot - Test Ce3t-IB

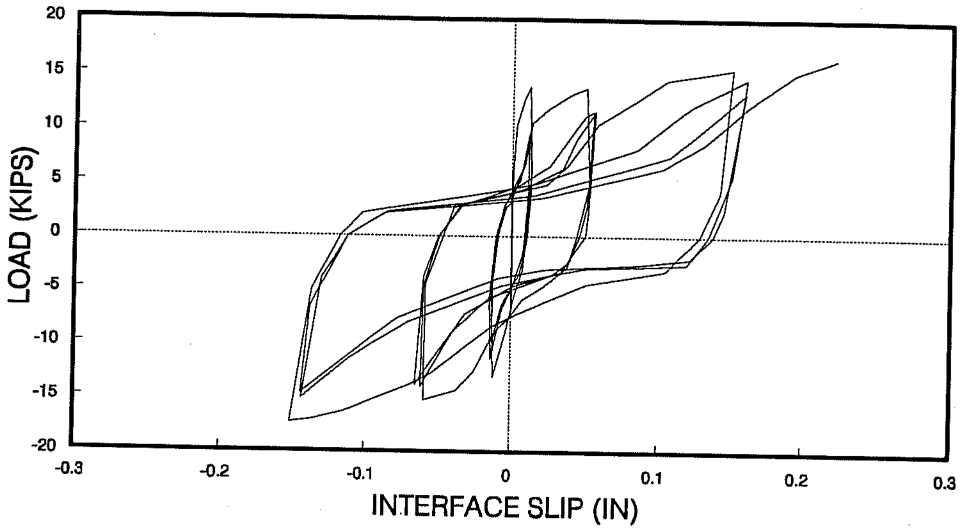


Figure 4.45 Load-slip plot - Test Ce3t-SP

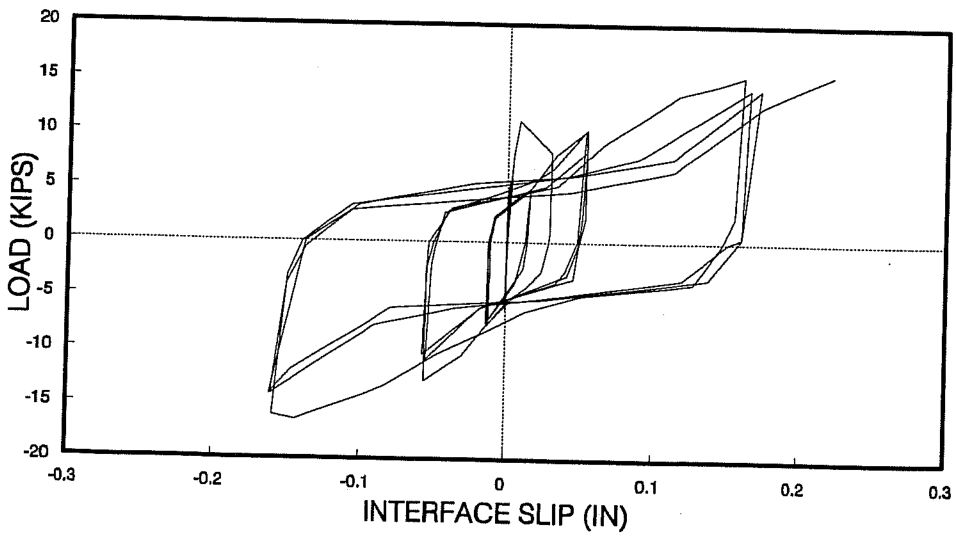


Figure 4.46 Load-slip plot - Test 2-Ce7t-SP

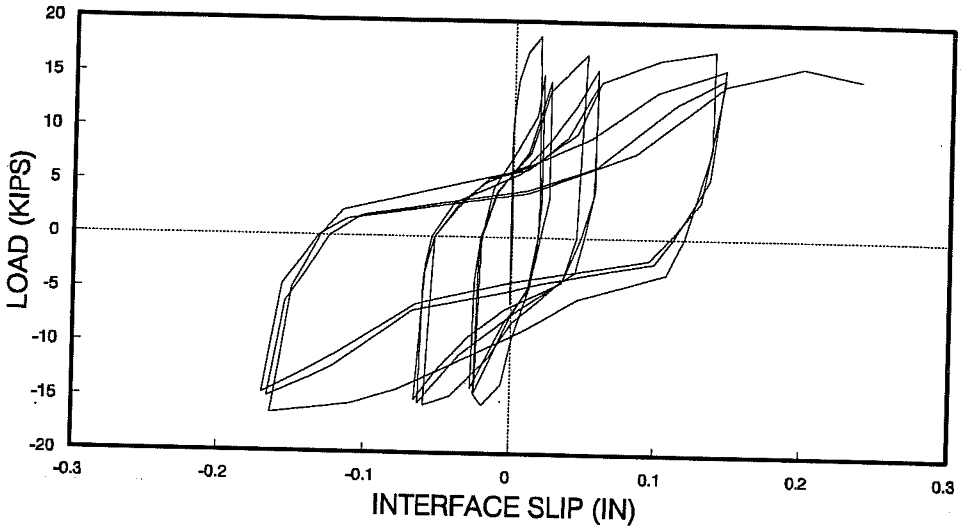


Figure 4.47 Load-slip plot - Test Ce3T-SP

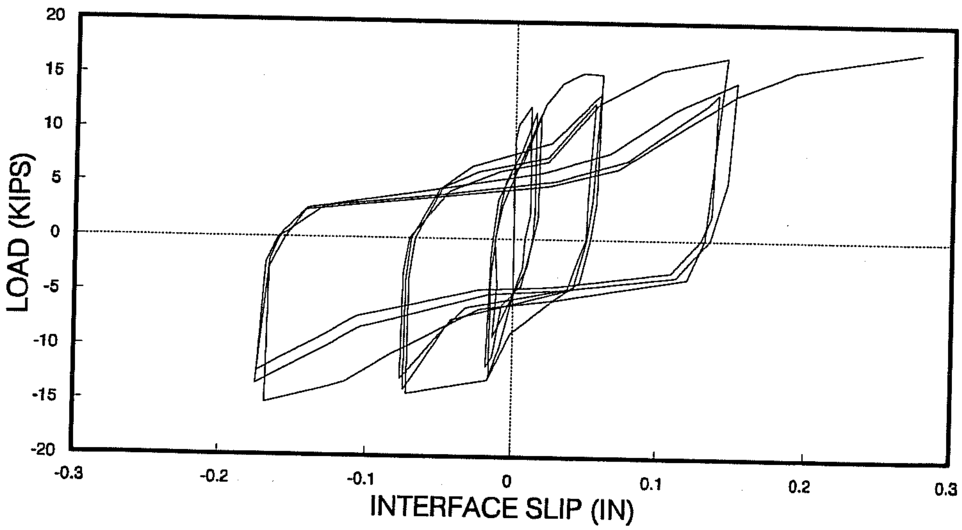


Figure 4.48 Load-slip plot - Test Ce7T-SP

4.5.1.2 Spring-loaded connections. Results of five cyclic tests of connections with spring washers are shown in Figs. 4.45 through 4.49. Spring washers helped in maintaining the initially applied clamping force throughout the different loading stages. As a result, less pinching was observed after load reversals. Furthermore, the area surrounded by the pinched loops was larger, which indicates higher energy absorption capacity. Hole clearance and clamping force did not have a significant influence on the rate of connection degradation for this test series.

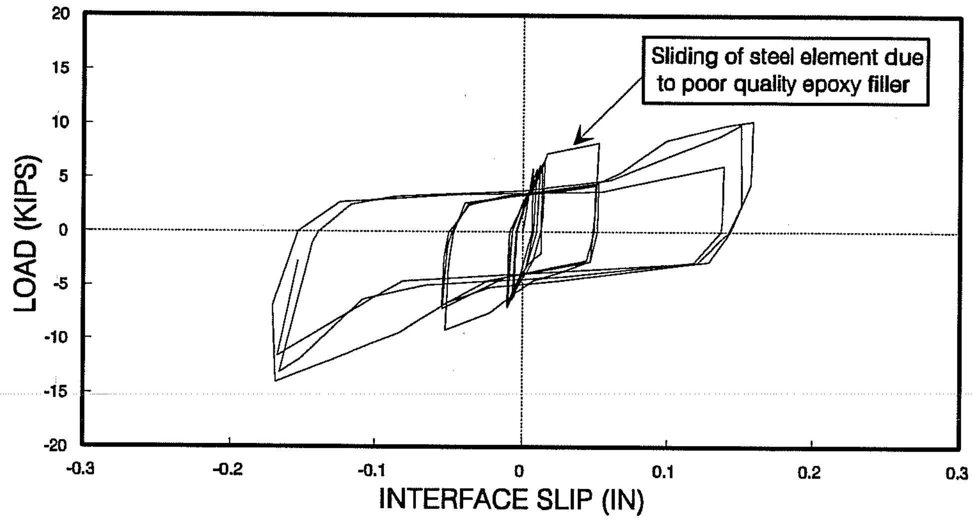


Figure 4.49 Load-slip plot - Test Ce7t-SP

Test Ce7t-SP was repeated because its performance was unsatisfactory as shown Fig. 4.49. Unhardened portions of the filler epoxy were observed upon removal of the nut and washer after the test was discontinued (See Fig. 4.50). Poor quality of the adhesive was likely the result of improper mixing. Since the epoxy in the annulus was not able to carry much load, behavior of this test approached the behavior of a plain spring-loaded connection. Load stages A and B were characterized by friction while bearing is only apparent in load stage C. Besides helping clarify connection behavior, this construction error emphasizes the importance of workmanship in performance of steel-to-concrete connections.

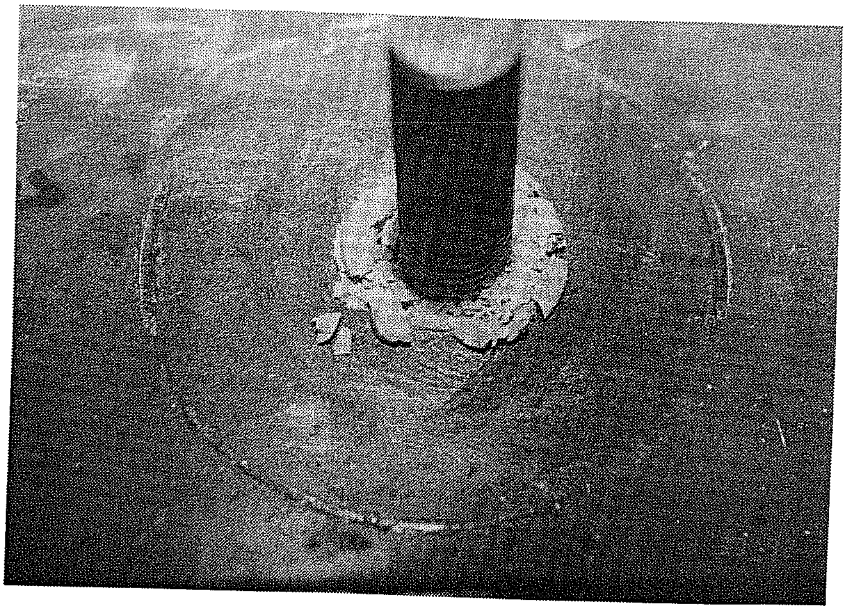


Figure 4.50 Unconfined, poor quality epoxy adhesive crushed under repeated cycles of loading - Test Ce7t-SP

4.5.1.3 Welded nut test. The load-slip response of the welded nut connection, shown in Fig. 4.51, was different from all others. Since the specimen was hand-tightened and no filler materials were included, connection stiffness in this case corresponds approximately to the stiffness of the anchor bolt fixed at one end by the welded nut, and at the other by the concrete. The fixity provided by welding the nut forced the anchor bolt to bend in double curvature below the steel element, as illustrated in Fig. 4.52. Examination of the top portion of the bolt after the test revealed that failure of the anchor bolt was characterized by kinking (See Figs. 4.52, 4.56 and 4.57).

Stiffness degradation at large displacements is mainly due to concrete damage under anchor bolt bearing (See Figs. 4.53 through 4.55). More crushing was observed in this test than in any other test, hence performance of this detail would be highly dependent on concrete quality. The concrete crushing zone extended over a depth of 1/2 in. after loose material was removed.

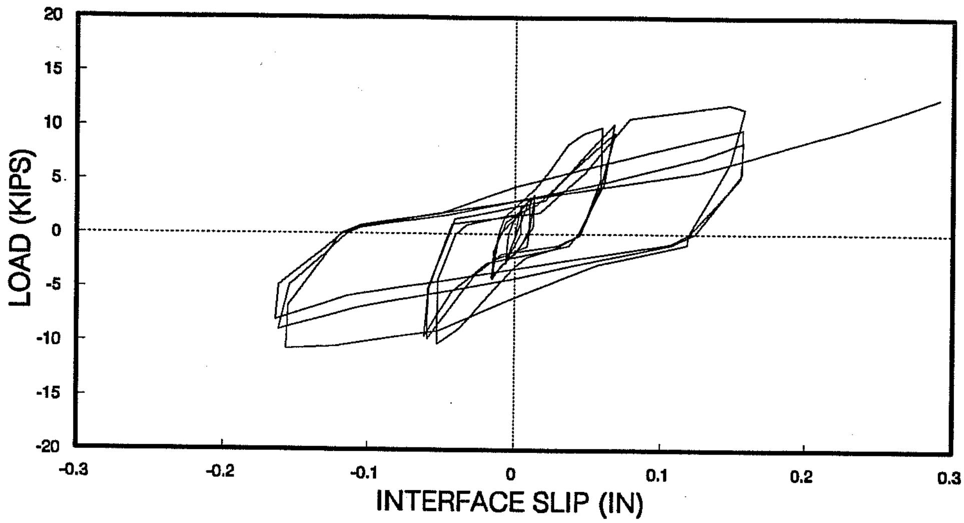


Figure 4.51 Load-slip plot - Test CN3h-WN

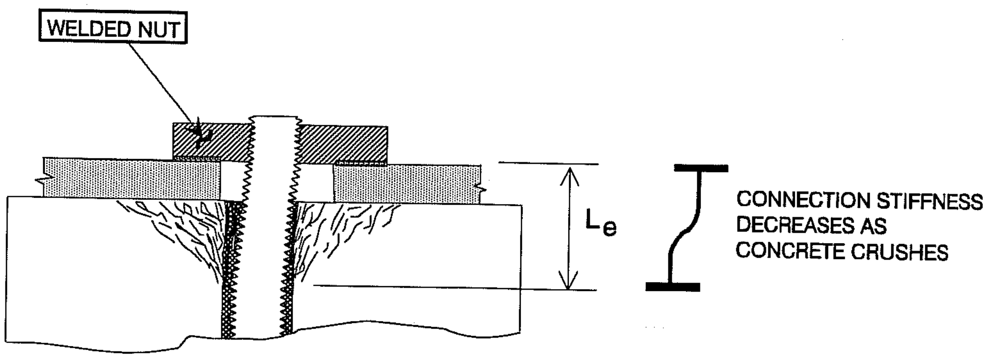


Figure 4.52 Schematic of behavior of welded nut connection

Figure 4.53 Crushing of concerte around bolt hole - Test CN3h-WN

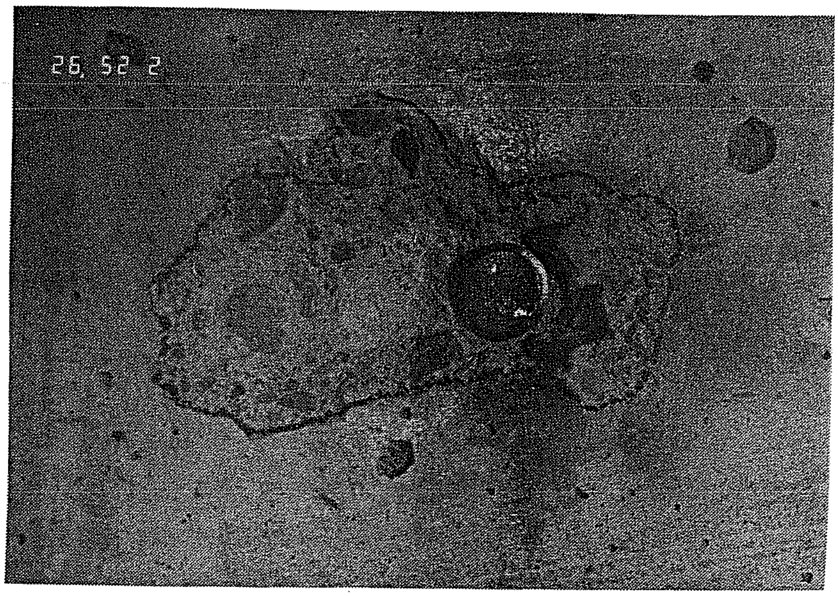


Figure 4.54 Concrete crushing zone after removal of loose material - Test CN3h-WN

Figure 4.55 Crushed concrete thrust into annulus of steel element by excessive bolt deformation - Test CN3h-WN

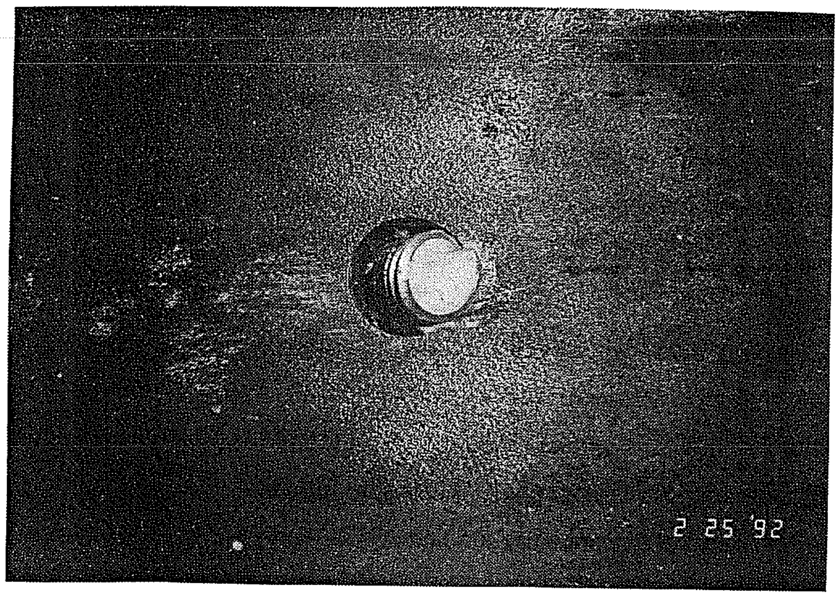


Figure 4.56 Anchor failure by kinking after bending in double curvature - Test CN3h-WN (viewed from bottom of steel section)

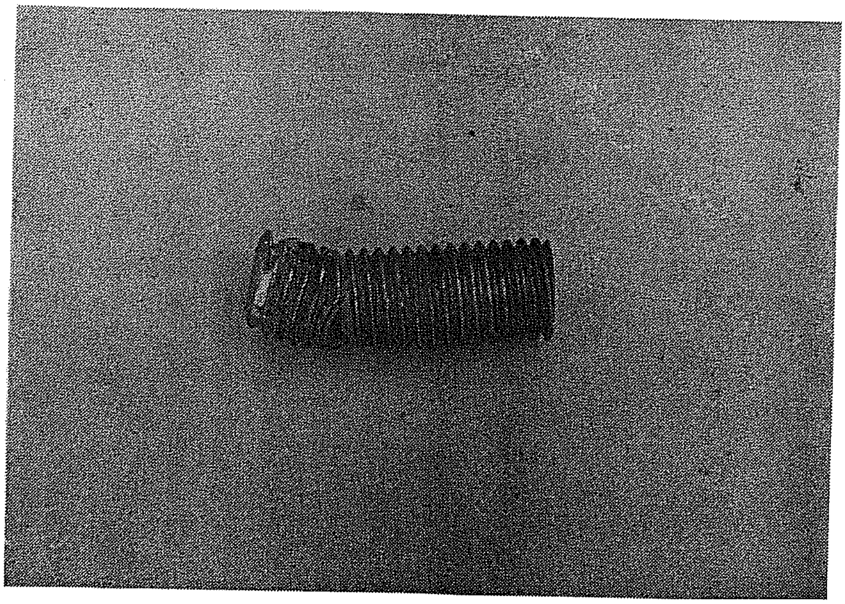


Figure 4.57 Top portion of failed anchor bolt - Test CN3h-WN

4.5.2. Envelopes of response. The peak loads obtained from load-slip curves were used to construct failure envelopes for cyclic tests. Envelopes are helpful in comparing stiffness of the different connections. Furthermore, the effect of repeated cycling on connection stiffness is studied by overlaying envelopes of response with load-slip plots from monotonic tests.

4.5.2.1. Tests with non-shrink grout. An examination of Figs. 4.58 and 4.59 shows that cyclic loading had a detrimental effect on the stiffness and capacity of connections with grout. The reduction in stiffness obtained in cyclic tests with grout is attributed to crushing of filler material with repeated cycling. As shown in Figs. 4.60 and 4.61, significant crushing of the grout in the annulus was observed after failure. The difference in stiffness increases proportionally with hole clearance since the amount of grout participating in connection behavior is greater (See Fig. 4.62).

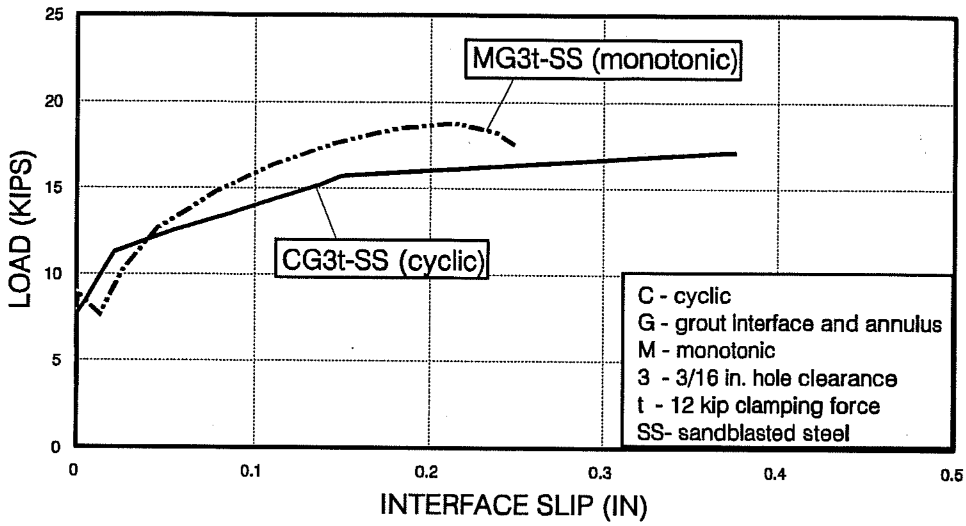


Figure 4.58 Comparison of response envelope and monotonic load-slip plot for the effect of cyclic loading (tests with non-shrink grout and 3/16 in. hole clearance)

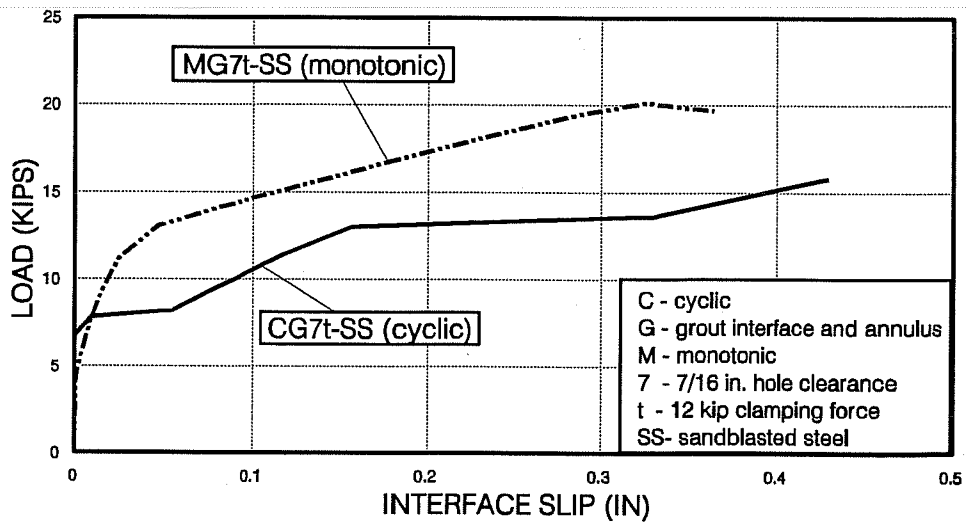


Figure 4.59 Comparison of response envelope and monotonic load-slip plot for the effect of cyclic loading (tests with non-shrink grout and 7/16 in. hole clearance)

Figure 4.60 Crushing of grout in annulus - Test CG7T-SS

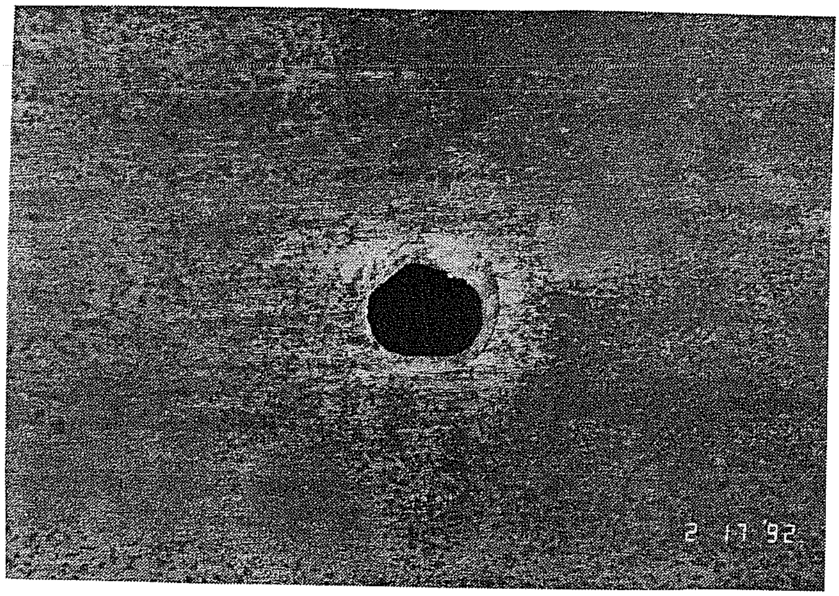


Figure 4.61 Oval shape of annulus after removal of loose material - Test CG7T-SS

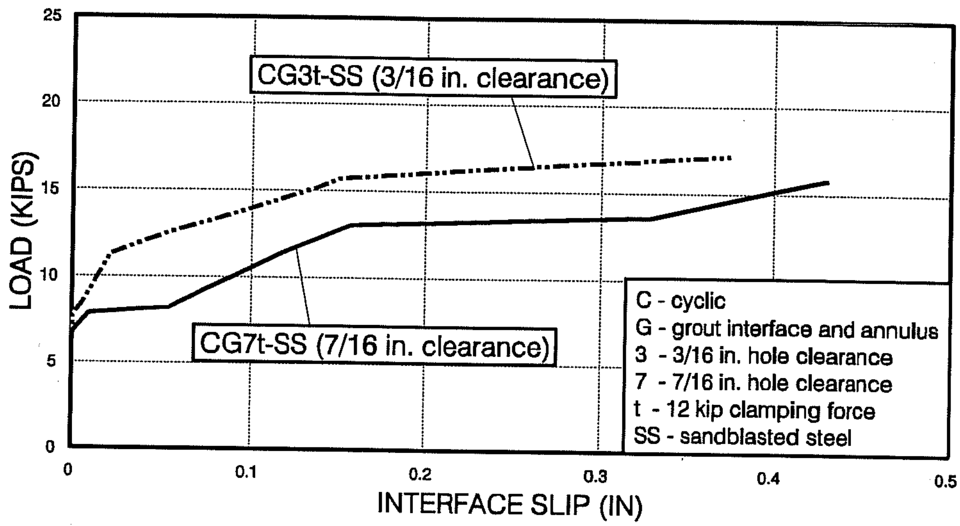


Figure 4.62 Response envelopes for the effect of hole clearance (tests with non-shrink grout with 12 kip clamping force)

Figures 4.63 and 4.64 show the influence of clamping force on connection stiffness. Specimens with 3/16 in. hole clearance and different levels of clamping behaved almost the same. However, clamping force increased connection capacity at all displacement levels for connections with 7/16 in. clearance. This is perhaps the result of better confinement of the grout filling the annulus provided by the higher clamping force.

The 1/4 in. thick grout interface of specimens CG3T and CG7T cracked, as shown in Figs. 4.66 and 4.69. Test CG3T was reproduced and the cement grout interface did not crack in the second test (See Fig. 4.67 and 4.68). An examination of Fig. 4.65 shows that interface cracking did not have an important influence on connection stiffness and capacity in this case. However, grout cracking increased the deformation capacity, which coincides with results of test MG3t-th.

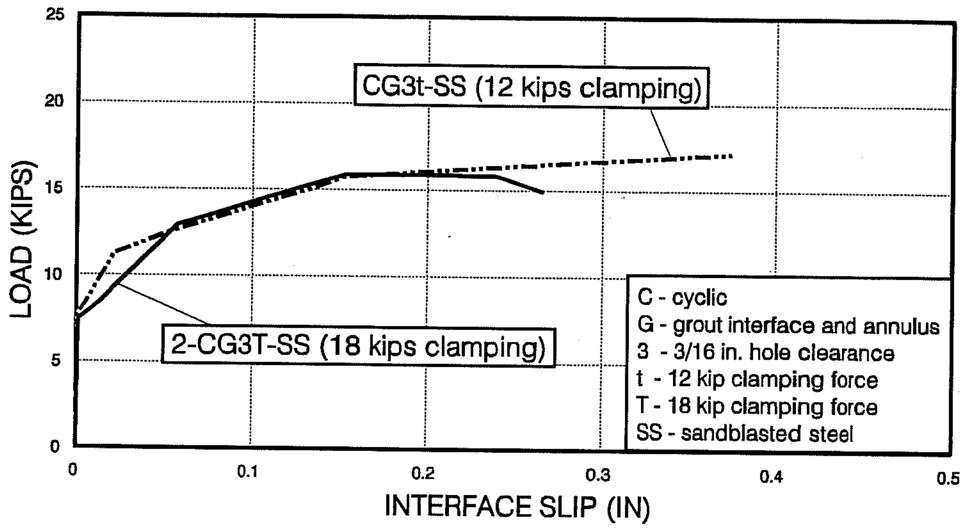


Figure 4.63 Response envelopes for the effect of increased clamping (tests with non-shrink grout and 3/16 in. hole clearance)

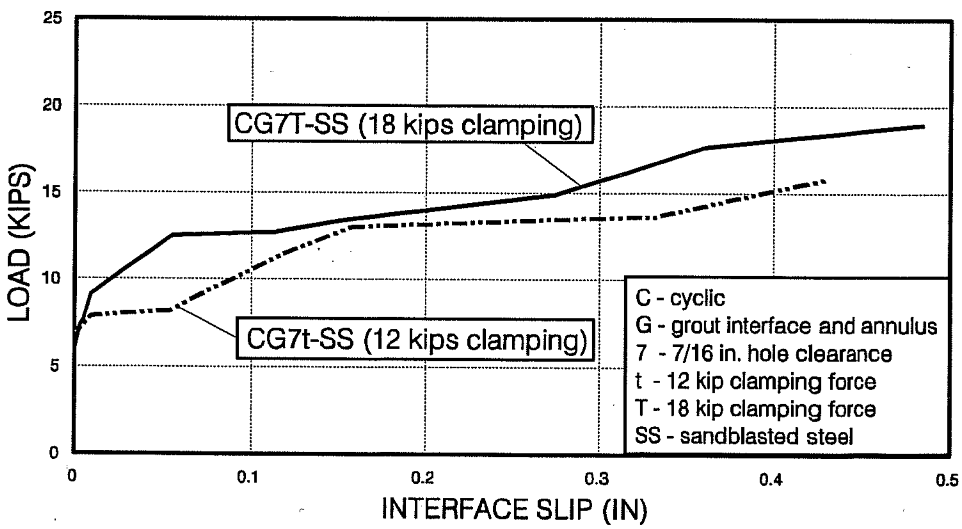


Figure 4.64 Response envelopes for the effect of increased clamping (tests with non-shrink grout and 7/16 in. hole clearance)

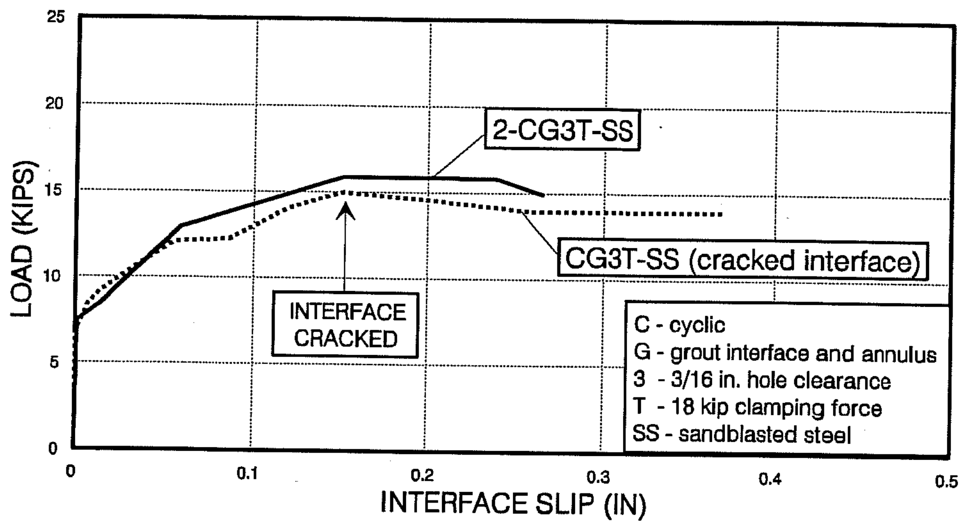


Figure 4.65 Response envelopes for the effect of grout interface cracking

4.5.2.2 Tests with epoxy-filled annulus - standard connections. Examination of Figs. 4.70 and 4.71 confirms the superior behavior of connections with epoxy as compared to similar connections with cement grout. The strength of specimens with an epoxy-filled annulus was higher than the strength of specimens with grout at all displacement levels for both hole clearances studied.

The influence of cyclic loading on connection stiffness is shown in Figs. 4.72 and 4.73. The difference in results from monotonic and cyclic tests is almost negligible for connections with 3/16 in. hole clearance. However, post-slip stiffness was reduced considerably by repeated cycling for connections with 7/16 in. hole clearance (See also Fig. 4.74). The reduction in post slip stiffness with increase in hole oversize is attributed to the larger amount of filler epoxy participating in connection resistance. As illustrated in Figs. 4.75 and 4.76, epoxy inside the annulus deforms plastically, allowing greater relative displacements between the steel element and the concrete member.

Figure 4.66 Cracking of interface grout - Test CG3T-SS

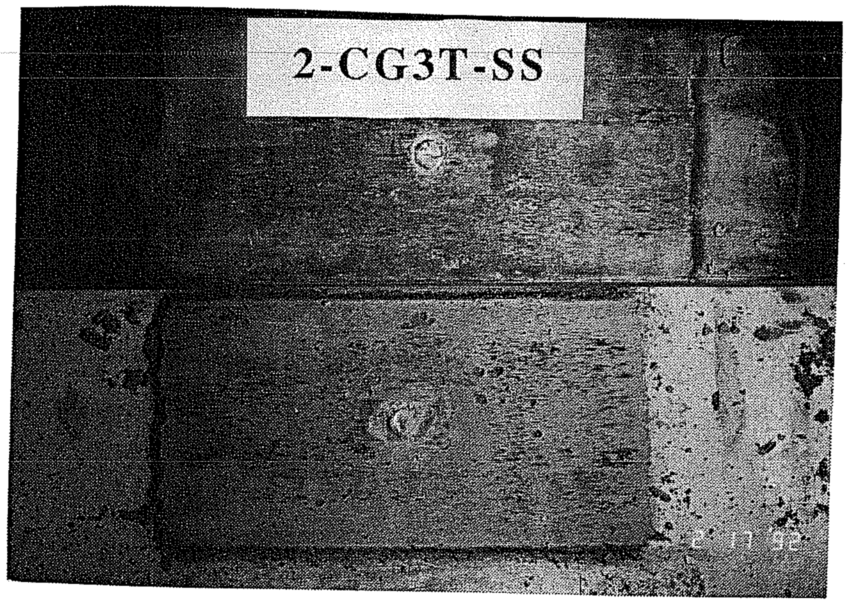


Figure 4.67 Grout interface after failure - Test 2-CG3T-SS

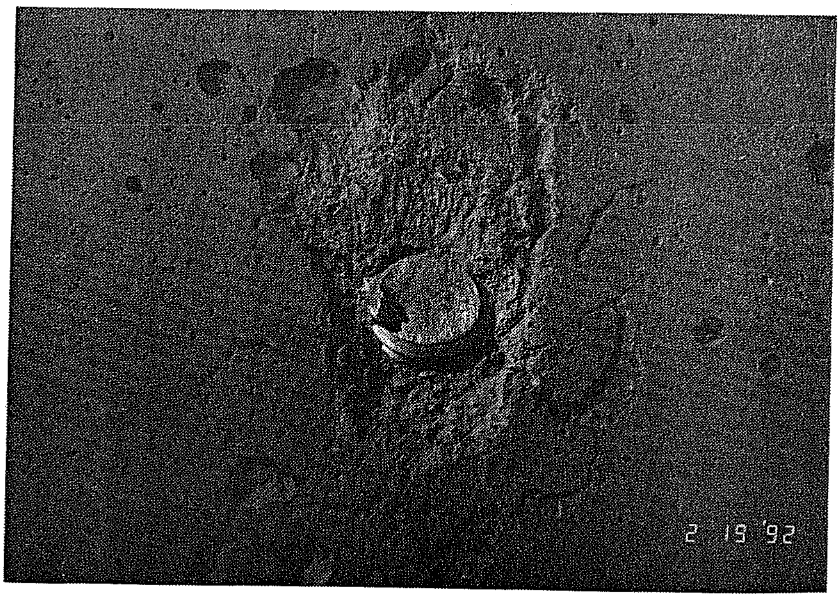


Figure 4.68 Crushing of grout around anchor bolt - Test 2-CG3T-SS

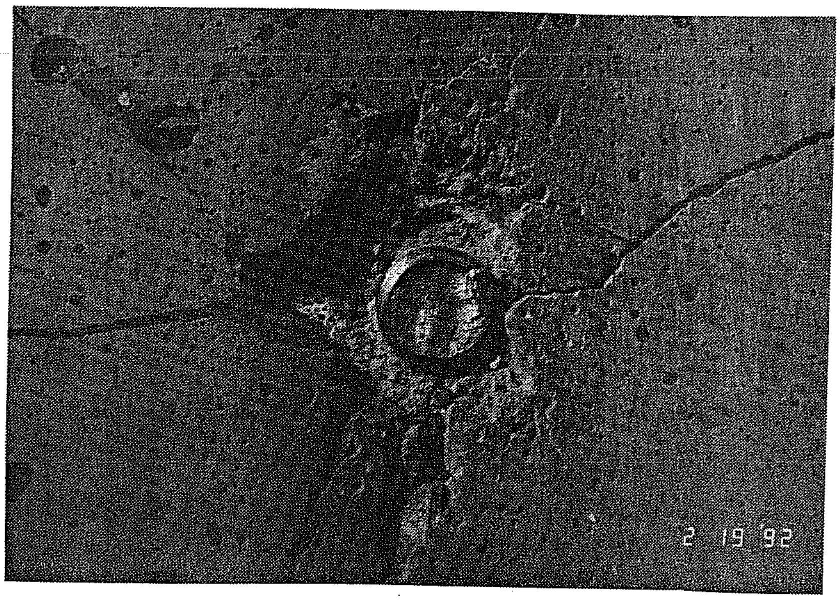


Figure 4.69 Interface cracking and grout crushing - Test CG3T-SS

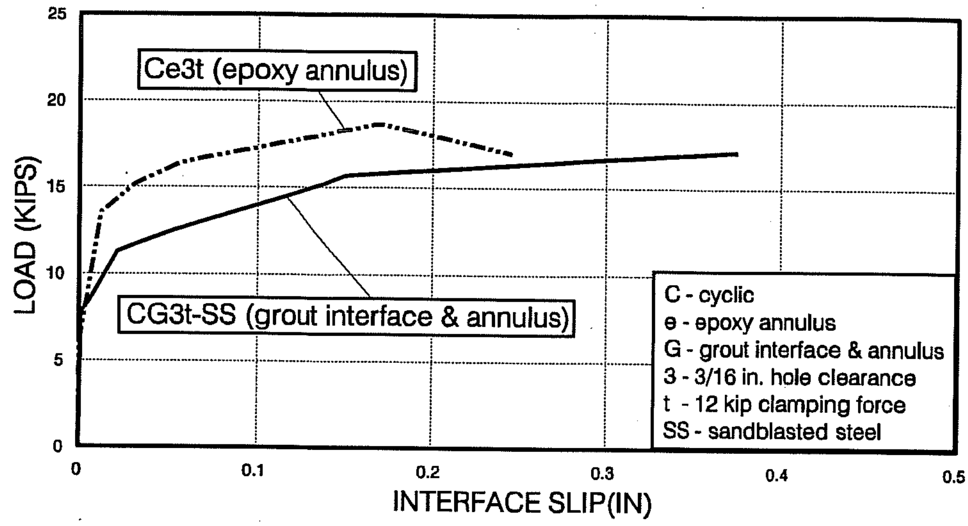


Figure 4.70 Response envelopes for test with epoxy-filled annulus and test with non-shrink grout having similar details (3/16 in. hole clearance)

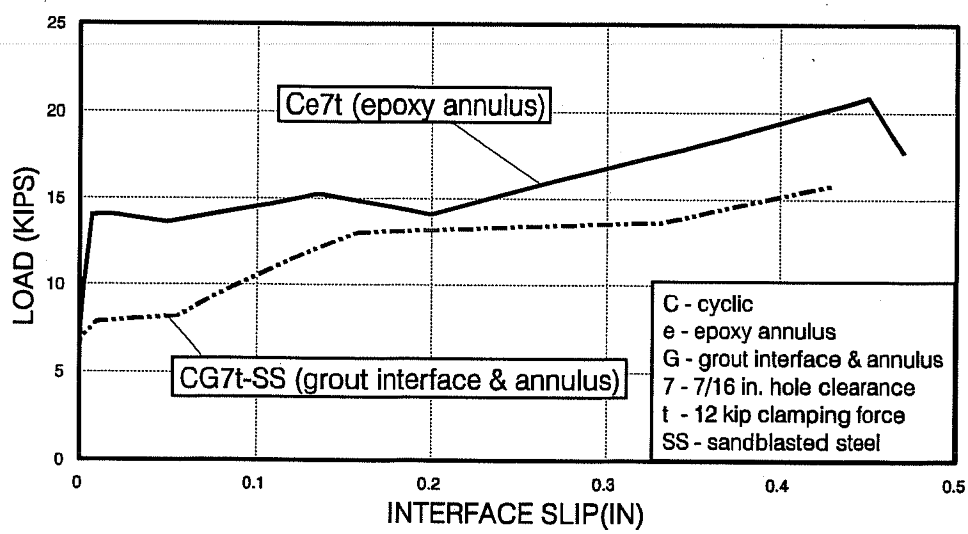


Figure 4.71 Response envelopes for test with epoxy-filled annulus and test with non-shrink grout having similar details (7/16 in. hole clearance)

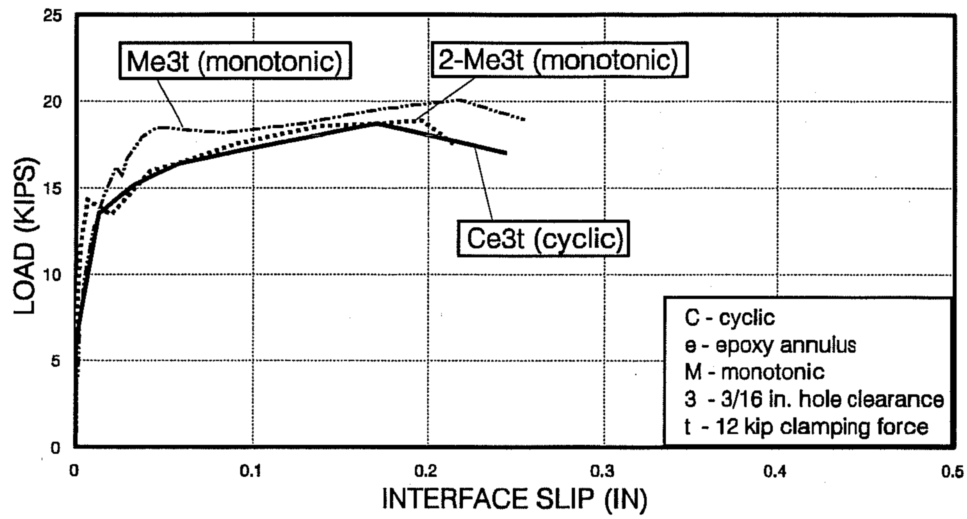


Figure 4.72 Comparison of response envelope and monotonic load-slip plots for the effect of cyclic loading (tests with epoxy-filled annulus and 3/16 in. hole clearance)

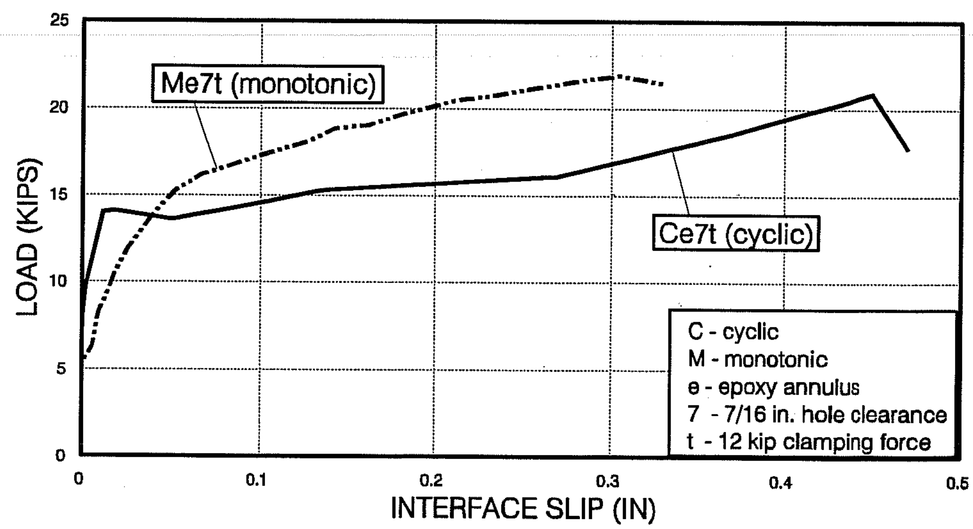


Figure 4.73 Comparison of response envelope and monotonic load-slip plot for the effect of cyclic loading (tests with epoxy-filled annulus and 7/16 in. hole clearance)

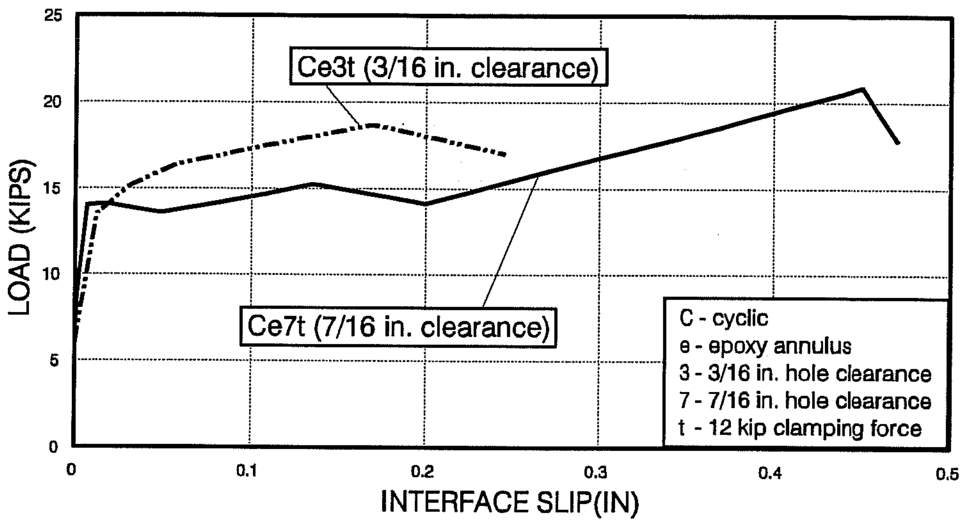


Figure 4.74 Response envelopes for the effect of hole clearance (tests with epoxy-filled annulus and 12 kip clamping force)

An increase in clamping force from 12 kips to 18 kips, provided no improvement in the behavior of a connection with 3/16 in. hole clearance. As shown in Fig. 4.77, the strength of the connection with 12 kips clamping force was higher than the strength of its companion with 18 kips clamping at all displacements beyond 0.03 in. It is believed that the reduced capacity of test Ce3T is the result of a more severe combination of shear and tension in the bolt.

Initial bolt inclination affected connection stiffness at different displacement levels. Figure 4.78. shows envelope curves of both sides of the response of test Ce3t-IB. The anchor bolt had an initial 5-degree inclination towards the north and the specimen was initially loaded towards the south. Examination of Figure 4.78 shows that behavior of connections with inclined anchors varies depending on the direction of loading, which agrees with results discussed in Sub-section 4.4.3.1.

Figure 4.75 Plastic deformation of adhesive in annulus (anchor bolt originally in center)

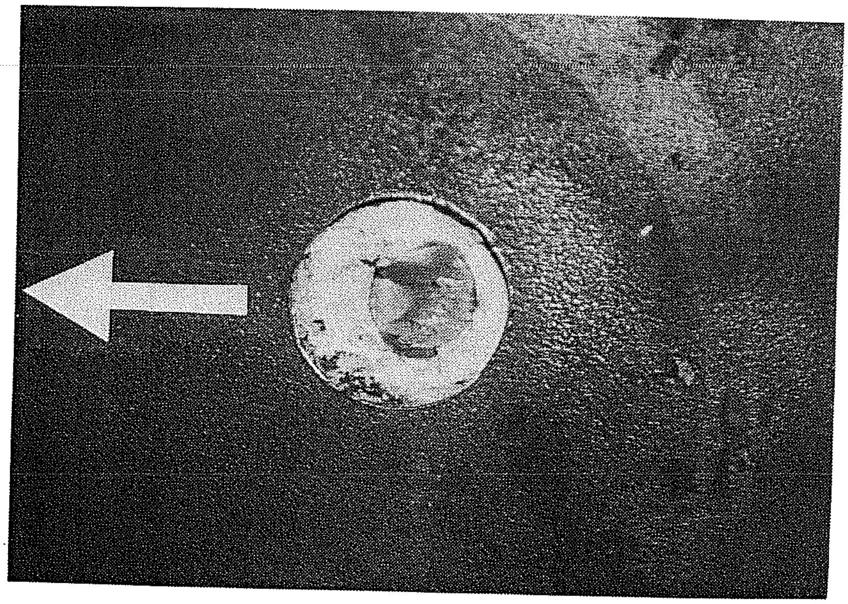


Figure 4.76 Plastic deformation of adhesive filling the annulus (anchor bolt originally in center) - closeup view

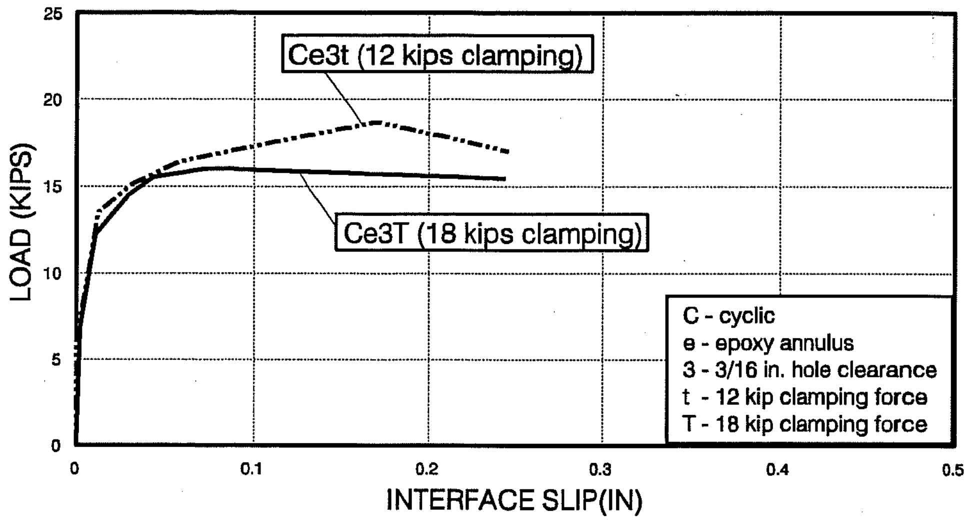


Figure 4.77 Response envelopes for the effect of increased clamping force (tests with epoxy-filled annulus and 3/16 in. hole clearance)

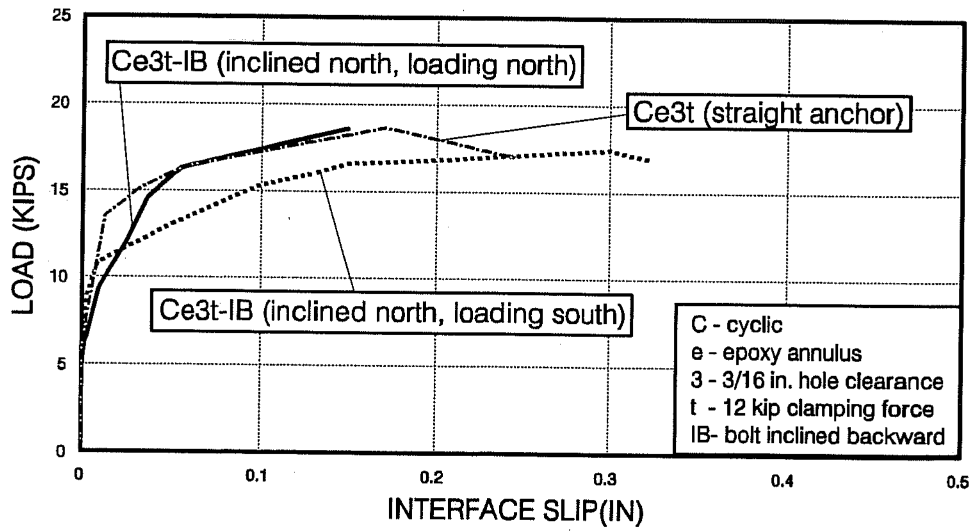


Figure 4.78 Response envelopes for the effect of bolt inclination (tests with epoxy-filled annulus and 3/16 in. hole clearance)

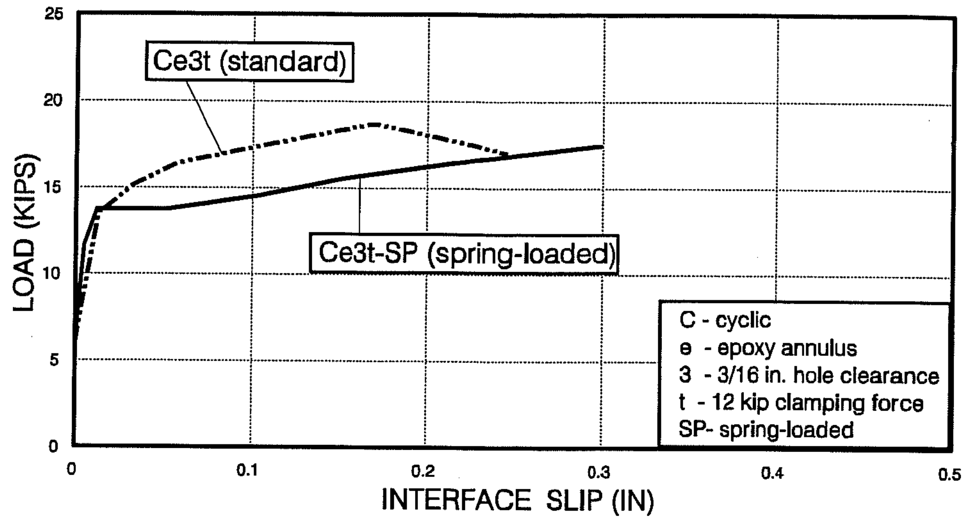


Figure 4.79 Response envelopes for tests with spring-loaded washer and standard nut and washer (3/16 in. hole clearance)

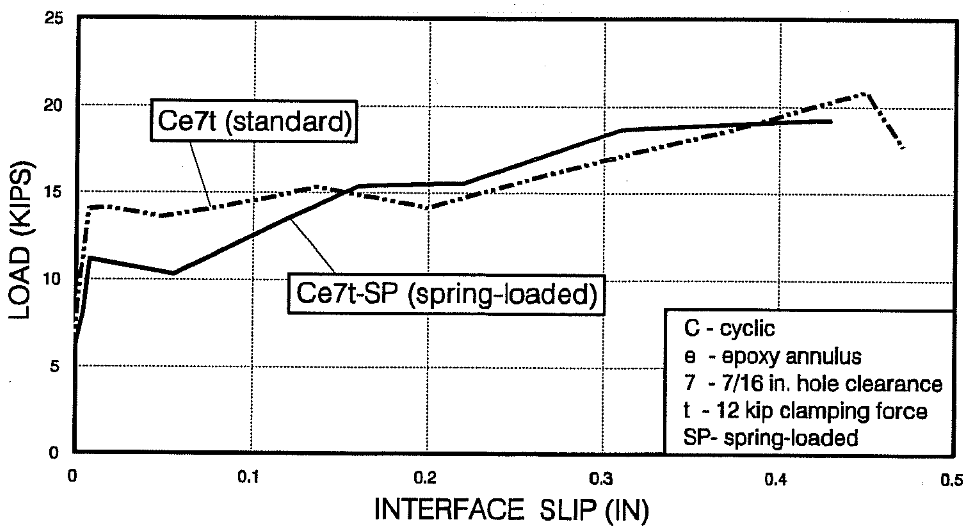


Figure 4.80 Response envelopes for tests with spring-loaded washer and standard nut and washer (7/16 in. hole clearance)

4.5.2.3 Tests with epoxy-filled annulus - spring-loaded connections. The difference in behavior between spring-loaded and standard connections torqued to the same level is illustrated in Figs. 4.79 and 4.80. Spring-loaded specimens exhibited less stiffness after first slip than standard connections. This difference was accentuated by increased hole clearance. It is believed that the observed decrease in post-slip stiffness is the result of increased plastic deformations in the epoxy filling the annulus. As indicated if Fig. 4.81, the spring washer, being of conical shape, does not confine the filler material like the standard washer does.

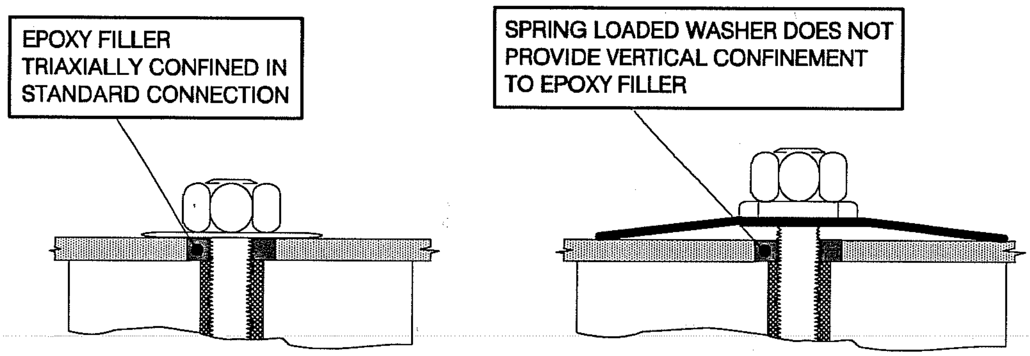


Figure 4.81 Schematic of state of confinement of filler with different washer types

However, it should be emphasized that this is a comparison of standard and spring-loaded connections with the same level of clamping. Whereas the clamping force in standard connections may relax because of creep in the retrofit anchors, spring-clamped connections should maintain a nearly constant clamping force with time.

As illustrated in Figs. 4.82 and 4.83, behavior of spring-loaded connections was considerably improved by increasing the applied clamping force. This improvement in response with increased clamping, not observed in tests with standard washers, is

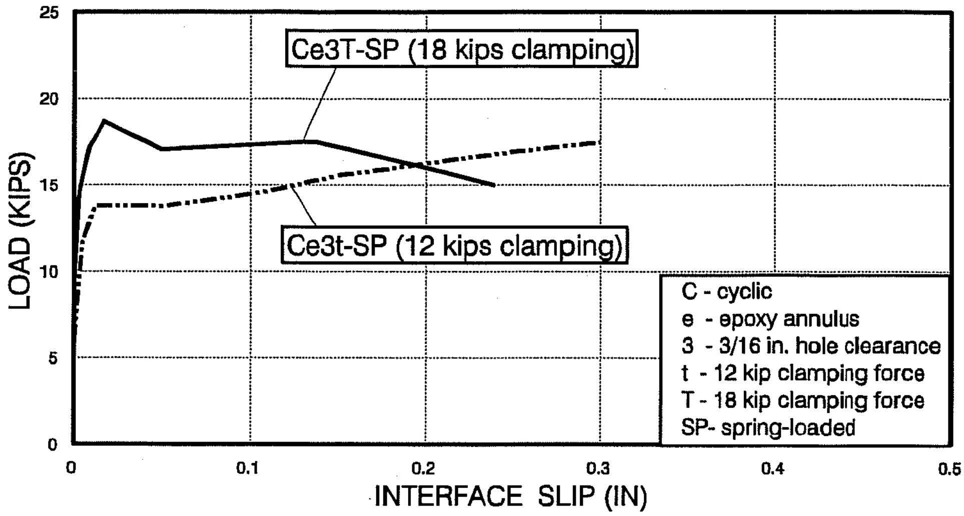


Figure 4.82 Response envelopes for the effect of increased clamping force (tests with spring-loaded washer and 3/16 in. hole clearance)

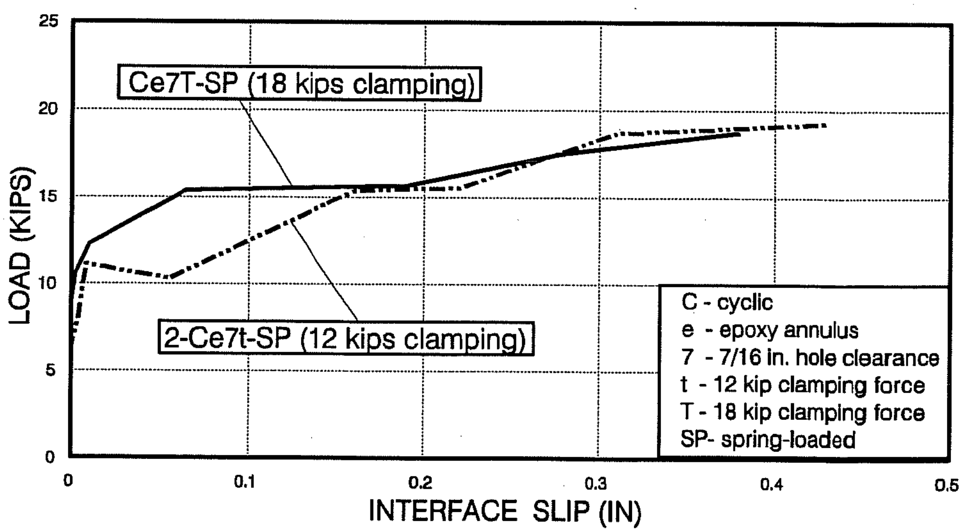


Figure 4.83 Response envelopes for the effect of increased clamping force (tests with spring-loaded washer and 7/16 in. hole clearance)

attributed primarily to the more effective transfer of the applied clamping force provided by the spring washer.

4.5.2.4 Welded nut test. Figure 4.84 illustrates the performance of the welded nut connection as compared to the plain attachment, and to a connection with filler epoxy. The welding detail improved post-slip behavior relative to the connection without adhesive fillers up to a displacement of 0.15 in. Nonetheless, the response of the attachment with filler epoxy remained superior. As shown in Fig. 4.84, test CN3h-WN had the same capacity as test Me3h before first slip since both connections were hand tightened. The observed difference in stiffness between the connection with epoxy and the welded connection, from first slip to 0.05 in., results primarily from the presence of adhesive in the annulus. It is believed that the load-slip response of the welded connection flattened beyond 0.05 in. slip due to concrete crushing and plastic deformation of the anchor bolt in reversed curvature (See Fig. 4.55 through 4.57).

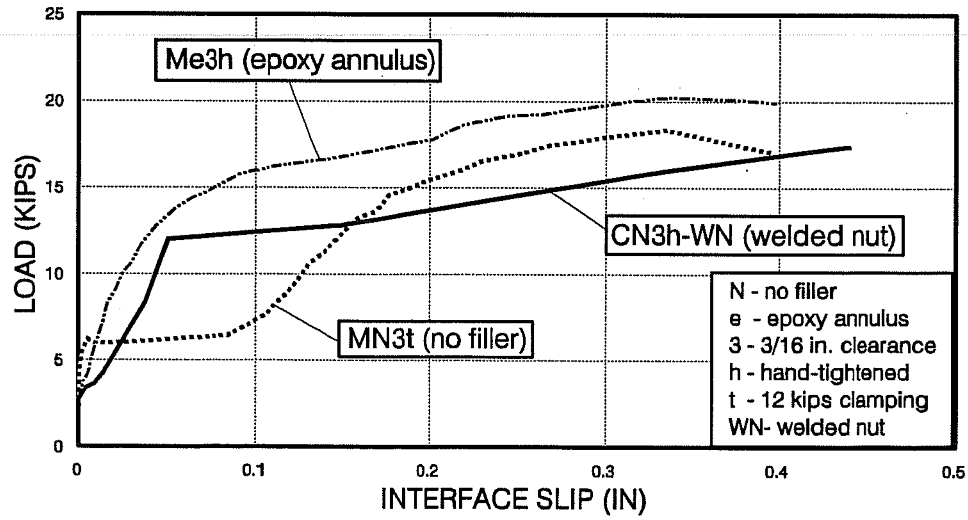


Figure 4.84 Response envelope for welded detail and load-slip plots for epoxy-filled and plain connections

4.6 Anchor Bolt Deformations

Bolt rotation above the steel element was monitored by two linear potentiometers in contact with the bolt as shown in Fig. 3.31. Rotation was calculated by dividing the difference between readings from the two potentiometers by the distance between them. Forces transferred to the bolt through the filler materials or by direct bearing against the web of the steel channel caused the bolt to rotate. As discussed by Wiener [45], a hinge developed in the bolt near the steel/concrete interface when subjected to large deformations. Hinging extended below the concrete surface as epoxy in the anchor hole and concrete crushed.

Typical rotation versus slip curves for monotonic and cyclic tests are presented in Figs. 4.85 and 4.86. Rotation-slip curves illustrate the following aspects of behavior:

1) Anchor bolt participation in response, and

2) Nature of bolt deformations.

A flat slope of the rotation-slip curve indicates either that the steel element is sliding relative to the concrete member and/or that the anchor bolt is deforming primarily in reversed curvature. Steeper rotation-slip curves indicate that the anchor bolt is bending in single curvature.

Bolt rotation data obtained agree well with load-slip results. The curve for test MN3t in Fig. 4.85 exhibited a negligible amount of rotation before the web of the channel bore against the bolt at approximately 0.1 in. slip. Results of tests with filler materials show the anchor bolt starts rotating before significant slip develops. All rotation-slip curves for connections with filler materials have a higher slope at all stages than the plain connection (more bending of bolt). In addition, increased hole clearance allowed for greater bolt rotation, which coincides with the fact that connections with greater oversize

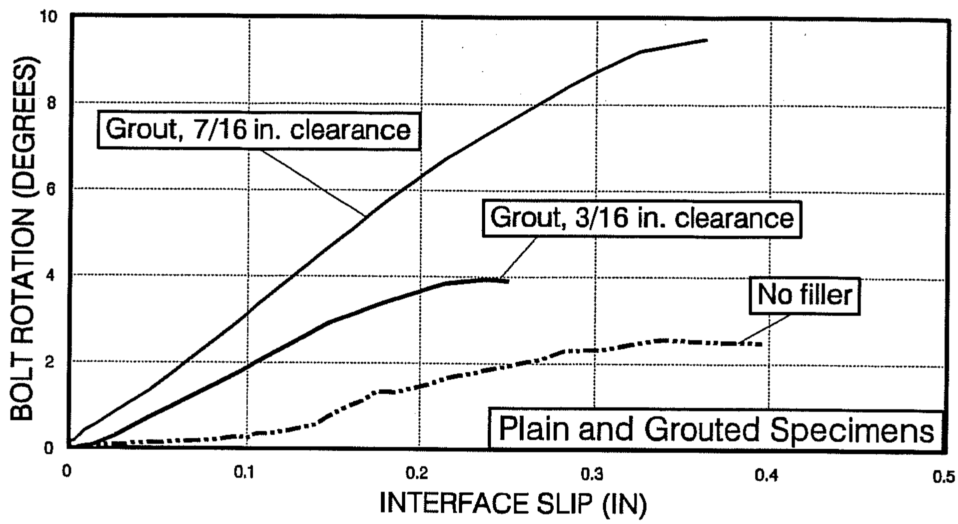


Figure 4.85 Sample rotation-slip plots for monotonic tests (plain test and connections with grout)

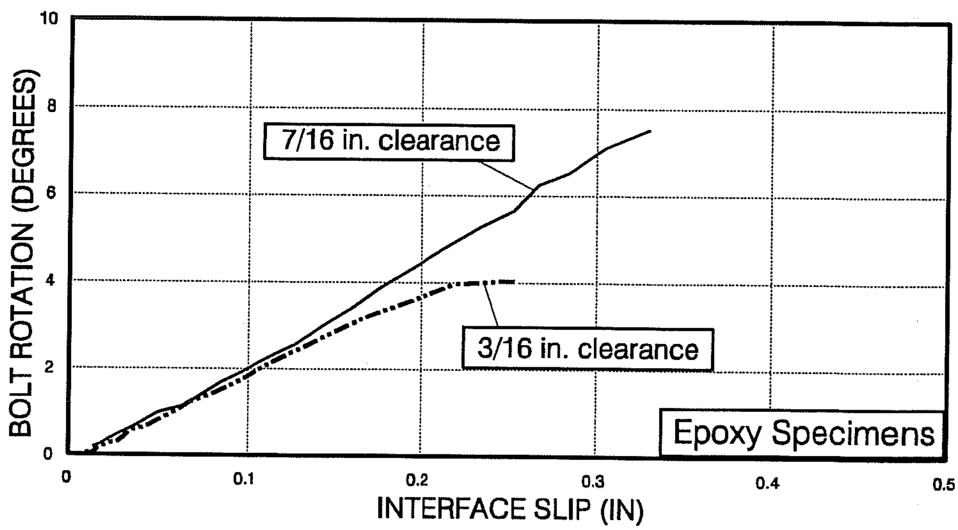


Figure 4.86 Sample rotation-slip plots for monotonic tests (connections with epoxy)

generally reached the peak load at larger displacements and had greater deformation capacity. Rotation data for standard connections incorporating grout and epoxy showed similar characteristics.

Figure 4.87 illustrates a sample rotation-slip curve for cyclic tests with standard a nut and washer. Distinct events characterizing the rotation-slip behavior are indicated in the figure. During the initial loading cycles, the bolts behave elastically giving a linear rotation-slip relationship. At larger deformations, anchor bolt rotation becomes more severe and permanent bending deformations become evident. Upon reversal of loading, the connectors first exhibit some elastic recovery. This event is followed by a low slope portion controlled by frictional slip. The last portion, showing increased bolt rotation, corresponds to bearing of the anchor after coming into contact with undamaged material. The slopes of the friction and bearing portions are governed by the particular details of each connection.

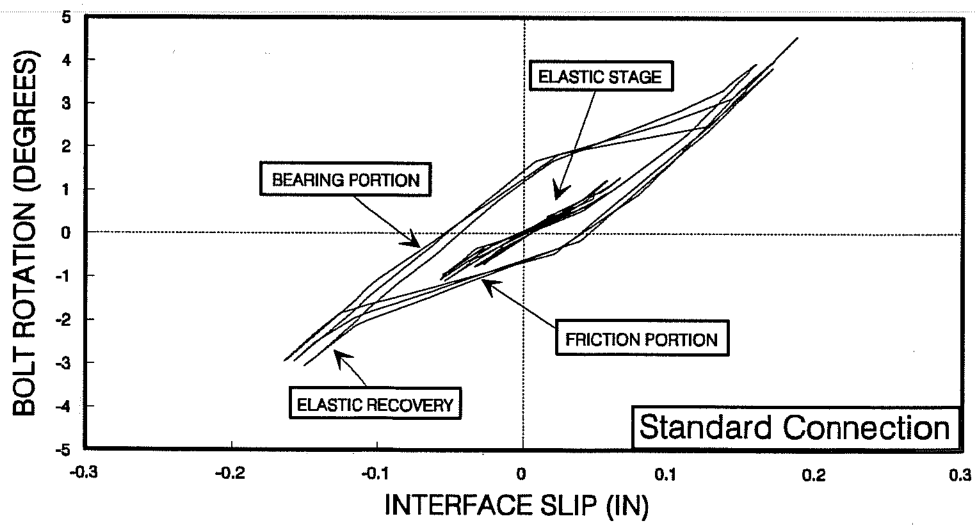


Figure 4.87 Typical rotation-slip plot for cyclic tests with standard washers - Test Ce7t

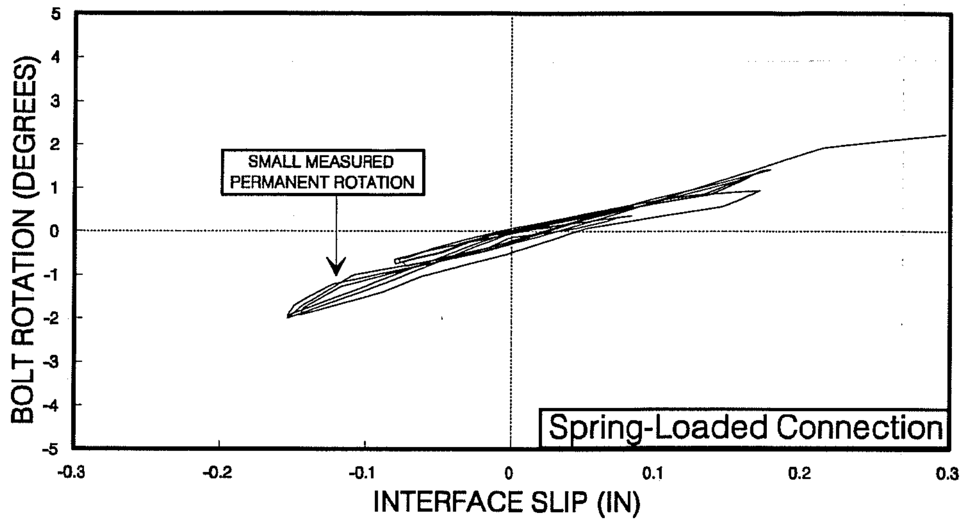


Figure 4.88 Typical rotation-slip plot for cyclic tests with spring washers - Test 2-Ce7t-SP

Sample rotation-slip behavior for the spring-loaded connection shows small anchor bolt rotations indicating substantial slip of the steel element relative to the concrete member (Fig. 4.88). This confirms earlier observations regarding behavior of connections of this type. Displacement is primarily characterized by frictional slip of the channel relative to the concrete block while epoxy in the annulus compresses. When the anchor bolt comes into bearing, it is subjected to a combination of shear and the nearly constant tension applied by the conical washer all around the nut. It is believed that the tensile force applied to the anchor bolt by the spring washer also contributes to keep it from rotating.

CHAPTER 5
EVALUATION OF RESULTS

5.1 Introduction

Test results obtained in this study are briefly evaluated as related to the design of steel-to-concrete connections. Early sections provide comparisons of load-response results with recommended design provisions. Recommended lower bound deformation capacities for analysis and/or design of multiple-anchor connections are presented and discussed in Section 5.3. Section 5.4 explains conceptually the behavior of a multiple-fastener connection through the use of a simple example. The last section discusses design and further research implications.

5.2 Capacity Before First Slip

5.2.1 Tests with non-shrink grout. The first-slip capacity of steel-to-concrete connections incorporating non-shrink grout in the interface is controlled by the frictional capacity at the steel and grout surface. In a recent research study, Cook performed 44 friction tests on multiple-fastener steel-to-concrete connections, obtaining an average coefficient of friction of 0.43 [19]. Cook, Doerr and Klingner recommended a coefficient of friction of 0.40 with a reduction factor of 0.65 in a Design Guide for Steel-to-Concrete Connections issued by the Center for Transportation Research of The University of Texas [20]. The value recommended by Cook et al. is compared with the results obtained in this study to determine its applicability.

Examination of Fig. 5.1 shows that all the specimens with grout had a coefficient of friction (Load at first slip/Applied bolt preload) higher than the 0.4 coefficient recommended in Reference 19. A value of 0.4 for the coefficient of friction for attachments with a grouted interface is therefore conservative.

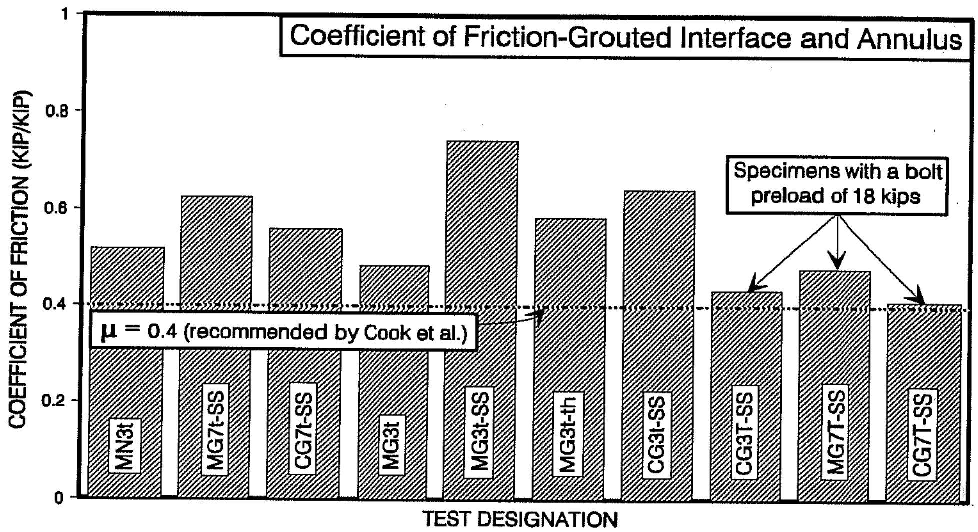


Figure 5.1 Comparison of coefficient of friction obtained for tests with non-shrink grout with the coefficient of friction proposed by Cook et al.

The following variables were examined separately to determine their influence on frictional capacity:

1) Effect of bolt preload: The level of clamping had a discernible influence on the coefficient of friction. While tests with an applied clamping force of 12 kips had an average coefficient of friction of 0.59, tests with a bolt preload of 18 kips had an average coefficient of friction of 0.43.

2) Effect of hole oversize: No significant differences were observed between the results obtained in tests with a 7/16 in. oversize (average coefficient of friction=0.52), and the tests with a 3/16 in. oversize (average coefficient of friction=0.54).

5.2.2. Tests with epoxy-filled annulus. The capacity before first slip of connections with an epoxy-filled annulus is controlled by a combination of friction and adhesion. Comparison of the measured loads at first slip for tests with an epoxy-filled annulus is shown in Fig. 5.2. The dotted line included in the figure corresponds with the capacity obtained using the coefficient of friction (0.4) used in the previous sub-section. The first-slip capacity of connections with epoxy showed more variability than the tests with non-shrink grout, and all tests demonstrated a first-slip capacity exceeding the calculated value.

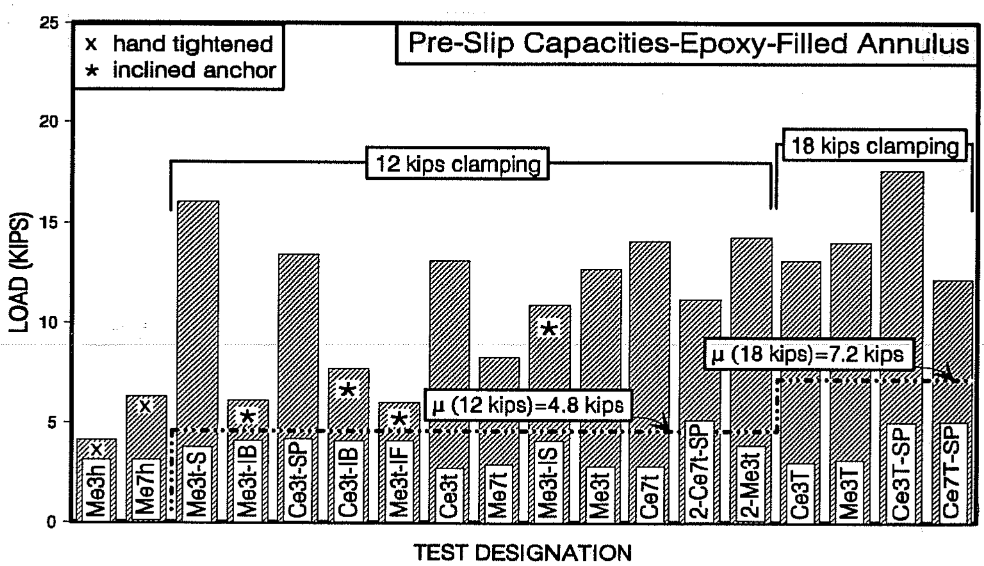


Figure 5.2 Comparison of first-slip capacities for tests having epoxy in the annulus with the coefficient of friction proposed by Cook et al.

The salient variables affecting the first-slip capacity of tests incorporating epoxy in the annulus include:

1) Effect of clamping level: Figure 5.2 demonstrates that hand-tightening should be avoided. The level of clamping imparted to the bolt is low and uncertain when the nut

is hand-tightened and hence the capacity before first slip is low. An applied clamping force of 60% of the yield capacity of the anchor bolt increased first-slip capacity by a factor of at least two for specimens with a 3/16 in. hole clearance, and by at least 30% for specimens with a 7/16 in. hole clearance as compared with hand-tightened connections. For specimens with standard washers and 3/16 in. hole clearance, an increase in clamping force above 60% of the tensile yield strength of the bolt did not produce a discernible additional improvement in first-slip capacity. However, the test with a spring washer and clamping force of 18 kips showed an increase of 26% as compared to the first-slip capacity of a similar connection with a standard washer.

2) Effect of bolt inclination: Besides hand-tightening, bolt inclination was the variable which most adversely affected the first-slip capacity of connections with an epoxy-filled annulus. The reduction in first-slip capacity was attributed primarily to an increase in short term clamping force relaxation. The average capacity before first slip of three connections with a 5 degree inclined anchor was equal to 6.6 kips (slip capacity was decreased by a factor of at least two as compared to tests with straight anchors and otherwise similar details).

3) Effect of hole clearance: The average first-slip capacity was reduced by 21% as the hole clearance was increased from 3/16 in. to 7/16 in. for a clamping force of 12 kips. This difference was mainly the result of an increase in short-term relaxation in the connections with a larger hole oversize.

5.3 Ultimate Capacity of Individual Anchor Bolts

According to different authors [10,24,29,30], the failure load of single-anchor steel to concrete connections controlled by failure of the anchor bolt depends on the bolt area and strength multiplied by a shear strength reduction factor α, as follows:

$V_n = \alpha A_s f_u$

where,

V_n = single-anchor connection nominal ultimate capacity
A_s = anchor bolt effective cross sectional area
f_u = steel tensile strength

Table 5.1 provides a summary of experimental results for the reduction factor α.

Table 5.1 Summary of previously published shear reduction factors

Reference	Interface conditions	Average reduction factor (α)
Kulak et al. [30]	steel/steel	0.62
TVA [24]	steel/grout/concrete	0.53
Burdette et al. [10]	steel/concrete	0.65
Klingner et al. [29]	steel/concrete	0.75

The American Concrete Institute Code Requirements for Nuclear Safety Related Structures and Commentary (ACI 349-85 and ACI 349R-85), provide shear/friction requirements for steel-to-concrete connections. The shear capacity is given by the equation

$V_n = \mu \, (A_s \, f_{ut})$,

which is based on the shear friction mechanism explained in Sub-section 2.3.2. The shear strength of the connection is therefore assumed to be a function of the tensile capacity of the anchor ($A_s \, f_{ut}$), and of the coefficient of friction between steel and concrete (μ).

The coefficient of friction μ, must be taken as follows:

"... μ = 0.7 for concrete or grout against as-rolled steel with the contact plane coincidental with the concrete surface...

... μ = 0.55 for grouted conditions with the contact plane between grout and as-rolled steel exterior to the concrete surface..." [5].

A coefficient of friction of 0.55 would apply to all the tests performed with non-shrink grout at the interface, and a value of 0.7 applies to all other tests. A reduction factor of 0.85 is recommended by ACI 349-85 if factored loads are used in the design.

Since the μ values proposed by ACI 349-85 are generally higher and less conservative than the average α values proposed by the authors listed in Table 5.1, the results obtained in this study are compared with those predicted by ACI 349-85 in Fig. 5.3. The peak capacity reached by each specimen with non-shrink grout exceeded the calculated ACI 349-85 ultimate strength value. However, as revealed in Fig. 5.4, two specimens with epoxy in the annulus and a clamping force of 18 kips did not achieve the ACI 349-85 calculated strength. The ultimate strength of connections having a 3/16 in. hole clearance and a bolt preload of 18 kips was reduced by 15 % for specimens with non-shrink grout and by 19 % for specimens with an epoxy-filled annulus. The use of an 18 kip clamping force is not recommended because its application reduced ultimate capacity and generally did not improve other aspects of connection performance. Other variables having a clear influence on ultimate capacity include:

1) <u>Interface thickness:</u> The ultimate strength of a specimen with a 1/2 in. thick grout interface was lower than the ultimate strength of all specimens with a 1/4 in. thick interface. The use of a 1/2 in. thick interface reduced ultimate capacity by as much as 24%.

2) <u>Type of fastening method:</u> Spring-loaded connections had a slightly lower ultimate capacity since the amount of tension in the bolt near failure was higher than for a

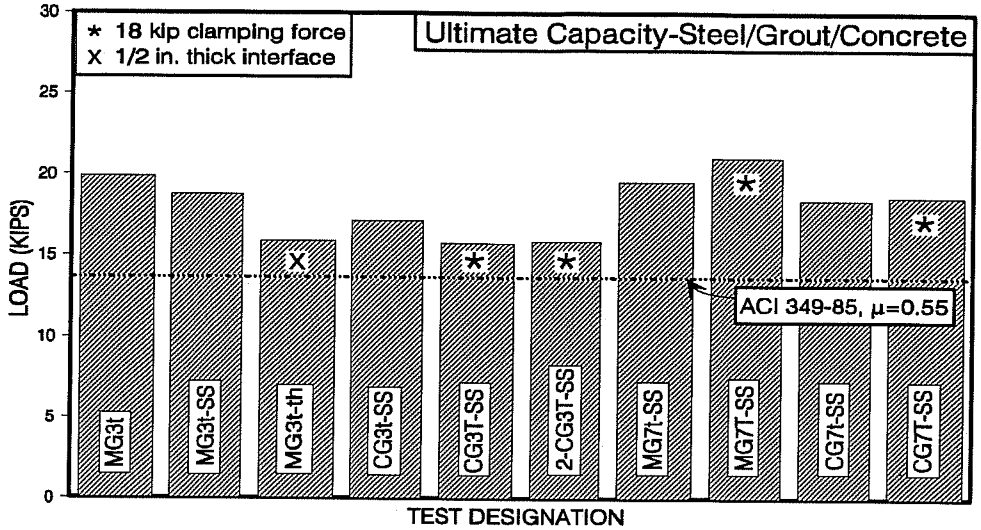


Figure 5.3 Comparison of measured capacities for tests with non-shrink grout and ACI 349-85 shear friction design strengths

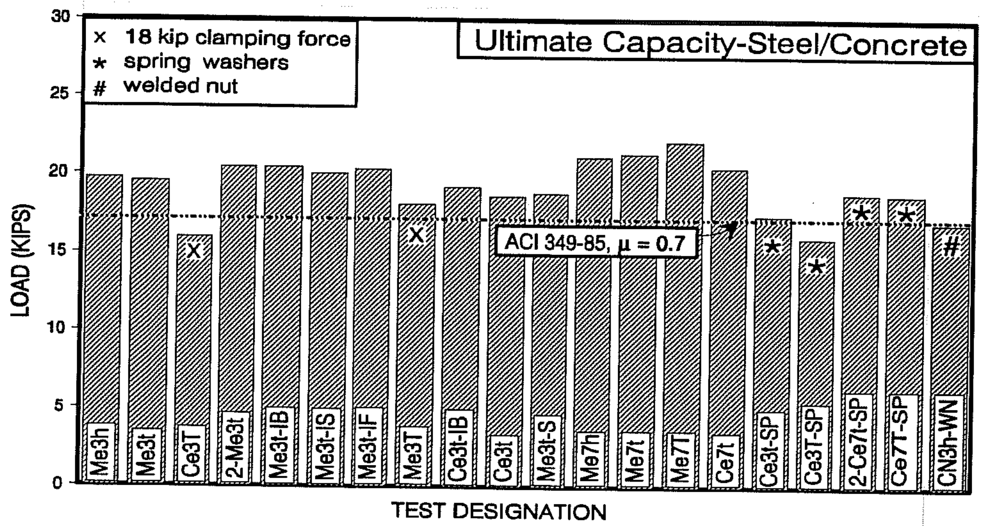


Figure 5.4 Comparison of measured ultimate capacities for tests without grout at the interface and ACI 349-85 shear friction design strengths

conventional nut and washer due to the presence of the spring washers. The higher tension in combination with shear resulted in a lower connection strength. In addition, the shear capacity of the welded fastener was reduced due to the more severe fixed condition imparted by welding the nut against the steel section.

3) Hole Clearance: The peak strength of specimens with epoxy was slightly enhanced by increased hole clearance (approximately 6% in the average). The larger hole oversize allowed the bolts to flex and rotate more, resulting in increased clamping on one side of the anchor bolt (due to the nut and washer rotating into the steel element) and an increase in frictional resistance near failure.

5.4 Deformation Capacity

As explained in Subsection 2.3.2, deformation capacity of connections is important when designing long joints. After reaching peak load, the highest loaded anchors must have enough deformation capacity to allow for redistribution of forces to other connectors.

Table 5.2 Lower bound deformation capacities

Hole clearance	3/16 in.					7/16 in.	
Filler or interface conditions	no filler	epoxy annulus	grout in annulus & 1/4 in. interface	grout in annulus & 1/2 in. interface	welded detail	epoxy annulus	grout in annulus & 1/4 in. interface
Lower bound deformation capacity (in.)	0.35	0.2	0.25	0.7	0.4	0.3	0.35

Table 5.2 presents the lower bound deformation capacity values obtained in this study. Lower bound here refers to a value equal to or smaller than the lowest interface slip recorded at failure for tests with the details specified in each column of the table. The results are separated according to the type of filler material or fastening technique and the hole clearance used. Deformation capacity was mainly affected by three variables:

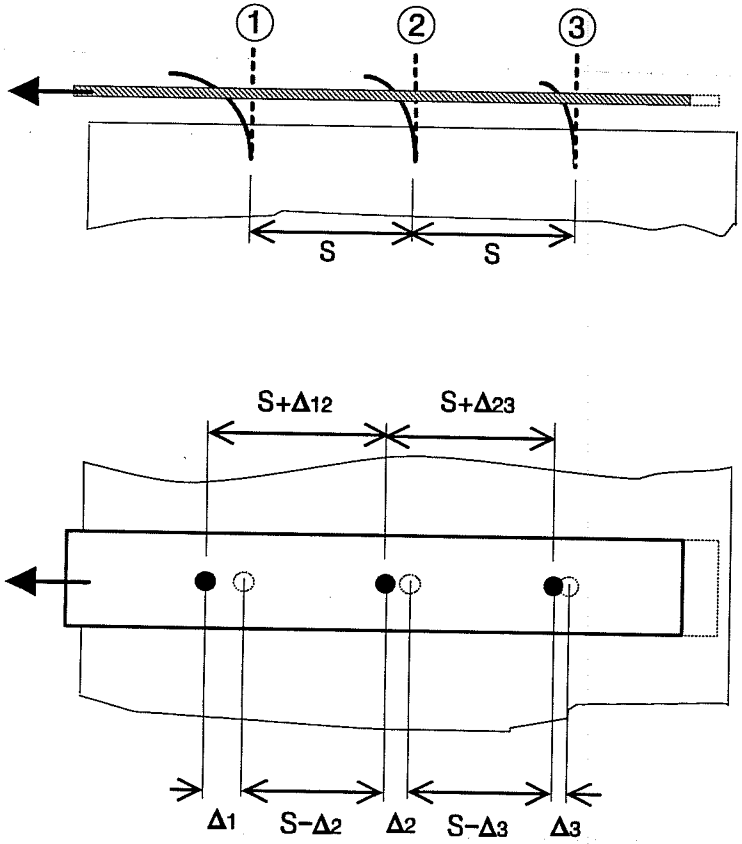
1) Type of filler material: Connections with 3/16 in. hole clearance and with non-shrink grout at the interface exhibited a slightly higher deformation capacity than similar specimens with an epoxy-filled annulus (See Table 5.2). Even though the test with no filler materials showed a relatively large deformation capacity, the value is not very reliable since a large portion of the relative slip occurred with no increase in load ("friction plateau" in Fig. 4.2).

2) Hole clearance: Increased hole clearance significantly improved the deformation capacity for both filler materials used. The lower bound deformation capacity was increased by 50 % for specimens with epoxy, and by 40 % for specimens with grout for the two hole clearances studied.

3) Interface thickness: The factor providing the most beneficial influence on deformation capacity was interface thickness. An increase in interface thickness from 1/4 in. to 1/2 in. improved the deformation capacity by a more than a factor of two. Since a thick epoxy interface is impractical, a 1/2 in. thick non-shrink grout interface is desirable when the design of a long steel to concrete joint is controlled by deformation capacity.

5.5 Estimate of Multiple-Anchor Connection Behavior

The load-slip behavior of a multiple-anchor connection may be predicted given the properties of the steel strengthening element, the spacing between connectors, and the empirical load-deformation response of individual anchor bolts. Figure 5.5 illustrates the



E: Modulus of elasticity of steel element
A: Cross-sectional area of steel element
Q: Applied load
Q_i: Load carried by anchor i
Δ_i: Deformation of anchor i
Δ_{ij}: Steel element elongation between anchors i and j

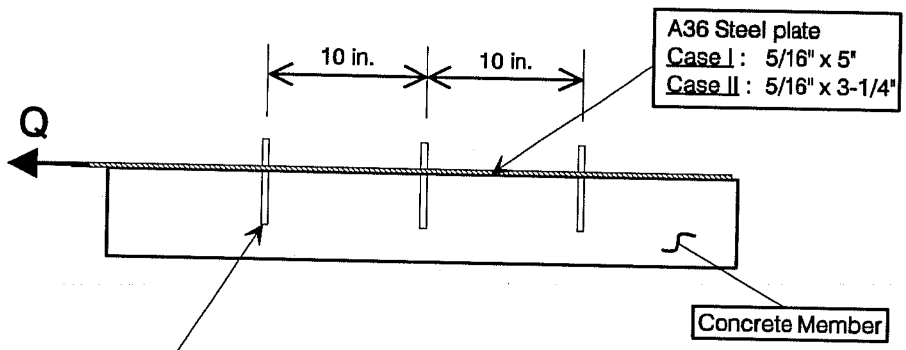
$$S+\Delta_{12} = \Delta_1+S-\Delta_2$$
or $\Delta_2 = \Delta_1-\Delta_{12}$

similarly

$$\Delta_3 = \Delta_2-\Delta_{23}$$

$$\Delta_{ij} = ((Q-Q_i)S)/(AE)$$

Figure 5.5 Schematic of distribution of deformations in multiple-anchor steel-to-concrete connection response of connections with several anchor bolts. The deformation of the first bolt is determined by the load it carries. However, the deformation of an interior connector (anchor 2 or 3 in this example), is equal to the deformation of the preceding connector minus the steel element elongation between the two bolts. Hence, the more flexible the plate, or the larger the spacing, the greater the difference in load transfer to bolts in a group. The distribution of forces among the different anchors can be determined for a given applied force Q by iterating assuming an initial value for the load carried by the first connector (Q_1). The ultimate strength can be found by setting the deformation of the first connector to its ultimate deformation.



3/4 " diam. epoxy-grouted anchors
(3/16 " hole clearance)

Technique A :

1/4" thick drypack filling all voids

Technique B :

Epoxy filling voids between holes in steel element and anchor bolt

Typical load-interface slip relationships			
Technique A		Technique B	
Slip (in.)	Load (kips)	Slip (in.)	Load (kips)
0	6	0	7
0.022	11.4	0.013	13.6
0.18	18.0	0.25	18.0

Figure 5.6 Details of sample multiple-anchor steel-to-concrete connection

Figure 5.6 shows the details of two three-connector attachments analyzed in this section for illustrative purposes. Connection behavior is predicted for both epoxy and non-shrink grout assuming the tri-linear single-anchor load-deformation relationships presented in the figure based on the results obtained in this study. The effect of localized bearing deformations need not be included since the thickness of the plates (steel elements) used in the analysis are equal to the web thickness of the channel sections used in the tests reported earlier.

Case I: In Case I, connection failure is controlled by failure of the first anchor bolt. Even though ultimate capacity is higher for Case I as compared to Case II (See Fig. 5.7 and 5.8), connection behavior is undesirable because the first connector fails before the steel plate is able to develop its yield load. The estimated distribution of forces among the three different anchor bolts for this case is shown in Figs. 5.9 and 5.10. The results of load distribution reveal that as deformations increase, the distribution of load among different bolts becomes more uniform due to non-linear behavior of the anchor bolts. However, in the elastic range, bolt load distribution is clearly unequal. This poses a serious question about behavior of long steel-to-concrete connections: How long could a connection be and still transmit load to the last anchor bolt ?

Case II: As illustrated in Fig. 5.8, connection behavior in Case II is controlled by yielding of the strengthening element. The load-slip plots obtained show that connectors with epoxy-filled voids would provide a more efficient means of transferring shear to the strengthening plate at early stages. In this case, the plate in the connection with non-shrink grout would have yielded at an interface slip three times higher than the connection with epoxy. After yielding of the strengthening element, the ductility of the connection is determined by the deformation capacity of the anchor bolts and could be estimated by an inelastic analysis of the connection as illustrated in Fig. 5.11, using simplified single-anchor load-deformation relationships based on the results reported herein and the load-elongation properties of the strengthening element. However, if the thickness of the steel element is different than that used in this test series, the analysis should also

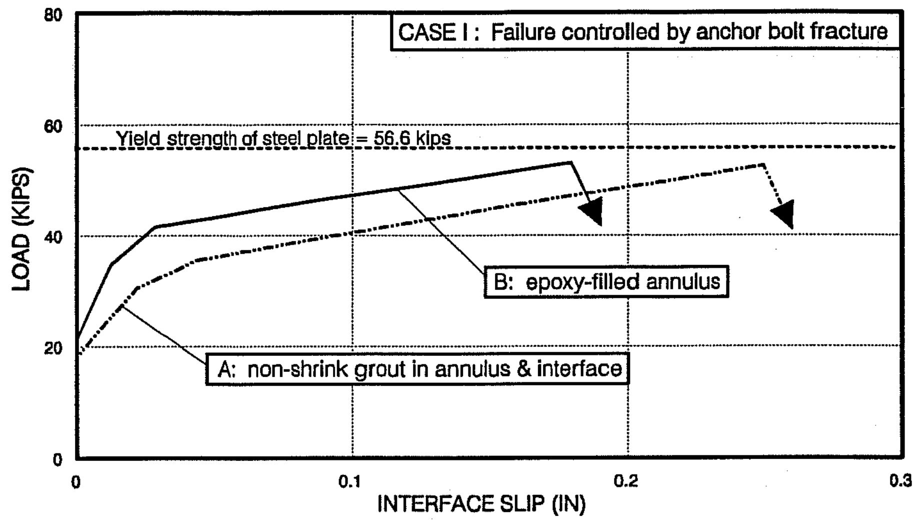


Figure 5.7 Load-interface slip response of analyzed multiple-anchor connection (Case I)

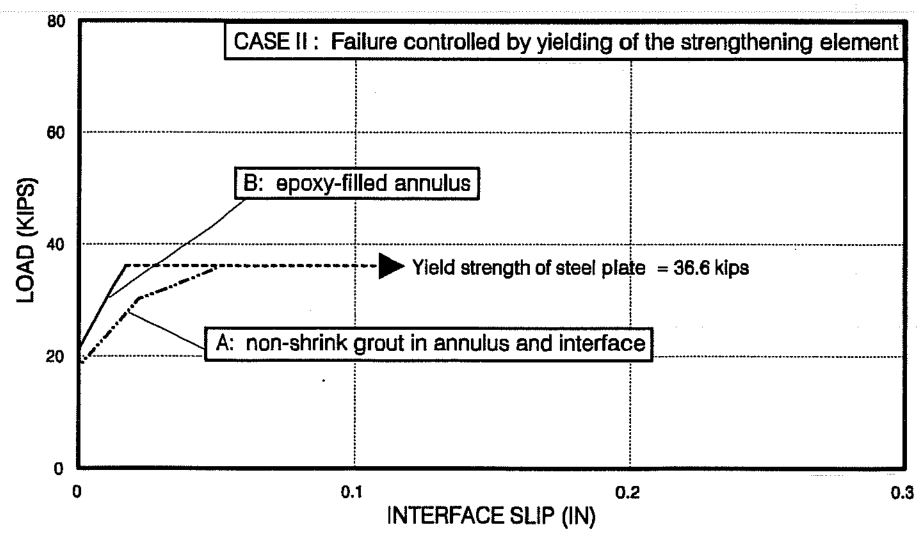


Figure 5.8 Load-interface slip response of analyzed multiple-anchor connection (Case II)

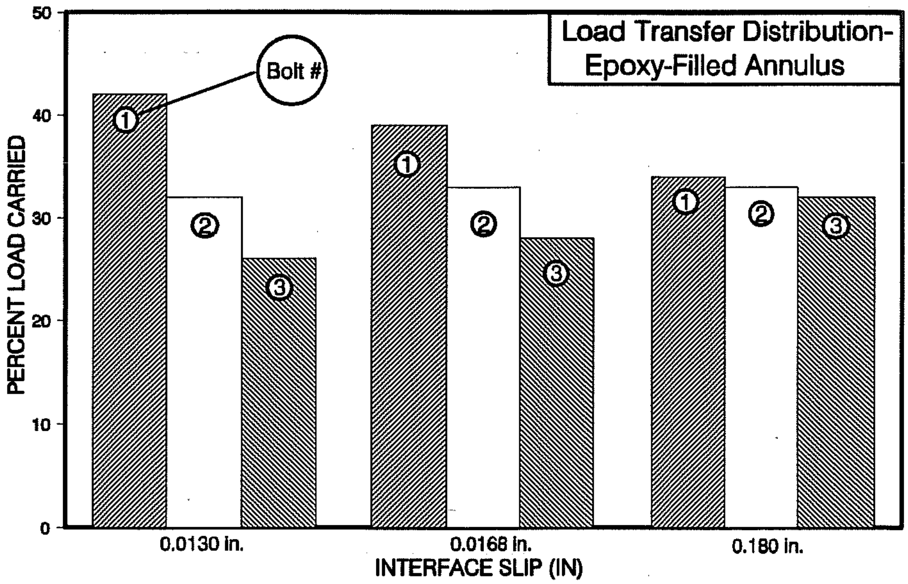


Figure 5.9 Estimated percentage of load resisted by bolts in sample connection with an epoxy-filled annulus (Case I)

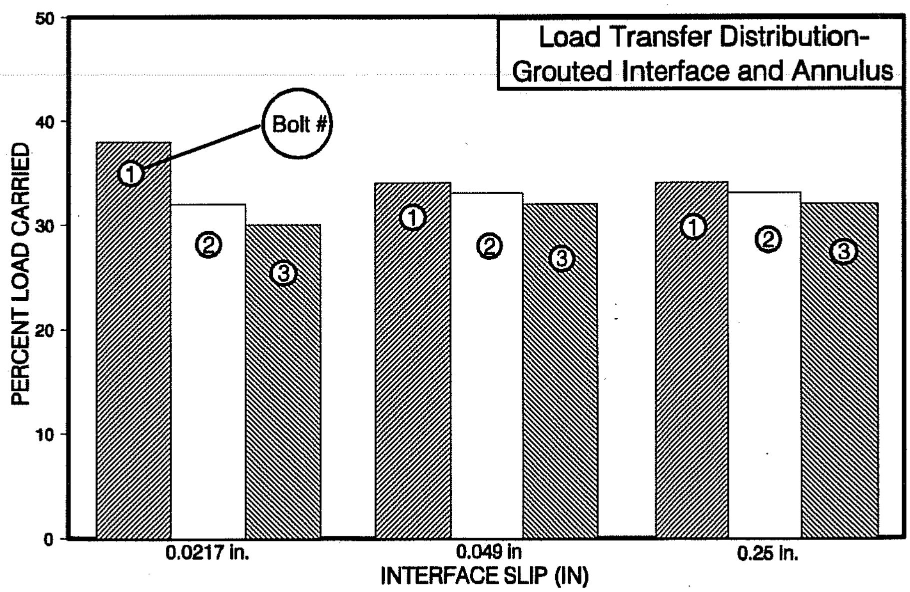
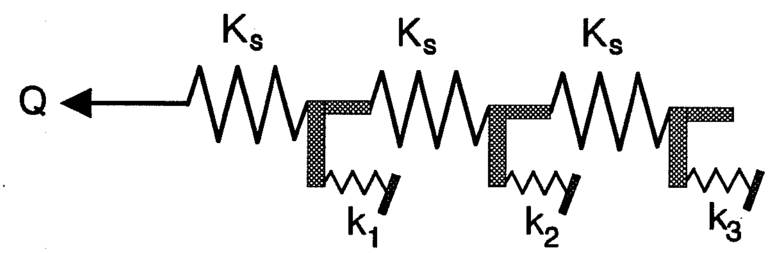


Figure 5.10 Estimated percentage of load resisted by bolts in sample connection with non-shrink grout (Case I)

include the effect of localized bearing deformations (especially if the steel section is thinner). Furthermore, the load-interface slip relationships obtained in this program are only applicable for the two hole clearances and anchor bolt size considered. For these reasons, the validity of any general analytical model should be verified by experiments on long connections.



K_s : Load-elongation properties of steel element

k_i : Load-slip response of individual connectors

Figure 5.11 Schematic of spring model for inelastic analysis of steel-to-concrete connections

5.6 Design and Research Implications

The particular details of each steel-to-concrete connection should be selected by the designer depending on the desired connection performance. For instance, collector or drag elements require high stiffness of the steel-to-concrete connections to transfer load to the bracing members with the least possible amount of slip. On the other hand, besides stiffness, deformation capacity is usually important for flexural strengthening of beams since the attached steel jacket will most probably yield, subjecting the anchor bolts to large deformations.

The comparisons made in Section 5.2 indicate that first-slip capacity of steel-to-concrete connections could be conservatively predicted by using a coefficient of friction of 0.4. However, the designer should not rely on first-slip connection capacity until the tension in the anchor bolts in service can be adequately predicted. The results of this study show that the following aspects should be taken into consideration by the designer whenever first-slip capacity is important:

1) Hand tightening the anchor bolts should be avoided,

2) The bolts should be aligned properly (no inclination). If inclination tolerances are allowed in construction, they must be taken into consideration in design calculations,

3) Smaller hole clearances are preferable,

4) Interface epoxy in combination with clamping force and appropriate surface preparation methods (sandblasting) substantially increase first-slip capacity, and

5) Spring washers may maintain the first-slip capacity with time by reducing long-term anchor bolt relaxation.

If anchor bolts are expected to undergo significant inelastic deformations, the use of a thick grout interface is recommended. The reduction in ultimate capacity and stiffness associated with a thick interface should not be neglected by the designer. Large hole clearance is another factor increasing deformation capacity. However, connection stiffness degrades at a faster rate with reversed cyclic loads when greater hole clearances are used.

The ultimate capacity of single-anchor steel-to-concrete connections is adequately predicted by the ACI 349-85 shear friction provisions. Applying a bolt preload beyond 60% of the yield capacity of the anchor bolt should be avoided to be on the conservative side of the ACI 349-85 shear friction equation. The number of anchor bolts required to develop the yield strength of the strengthening steel section can be determined by dividing the yield capacity of the strengthening element (multiplied by a factor of at least 1.25) by the nominal shear capacity of a single anchor bolt as calculated using the ACI 349-85 provisions. The recommended factor of 1.25 takes into consideration strain hardening and differences between nominal and actual strength of the steel element, and should insure yielding of the element prior to anchor bolt failure. Nevertheless, a simple and safe procedure for proportioning the steel-to-concrete connections (selecting the steel element cross section and bolt spacing) given the number of anchor bolts needed for shear transfer is not available. The solution of the response of multiple-anchor connections by the equilibrium and compatibility formulation presented in Section 5.5 is lengthy and obviously impractical for design purposes.

The approach included in the AISC specification for design of long steel shear connections consists of applying a reduction factor of 20 % in the design strength of bolts when

> "... splice tension members have a fastener pattern whose length, measured parallel to the line of force, exceeds 50 in., ..." [6].

Furthermore, Kulak et al. [30] recommend a reduction in the understrength factor (Φ) from 0.8 to 0.64 for connections longer than 50 in.

More research is needed to determine the necessary reduction factor(s) for design of long, multiple-anchor steel-to-concrete connections. A rational semi-empirical model, subjected to experimental verification, for the calculation of the characteristics of load-interface slip response of multiple-anchor connections seems to be the most promising approach for detailing long steel-to-concrete connections.

CHAPTER 6
CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary

An experimental program was implemented to determine the load-displacement behavior of steel-to-concrete connections incorporating several test variables. The following variables were studied in thirty six single-anchor steel-to-concrete connection tests: hole clearance, bolt preload, type of material used to fill the void between the anchor bolt and the steel element (non-shrink grout and epoxy), surface treatment, type of fastening method, bolt position and inclination, interface thickness, and effect of reversed cyclic loads. The test results consisted of load-slip relationships, maximum elastic and ultimate capacities, and envelopes of response for cyclic tests. Maximum elastic and ultimate capacities were compared with proposed design provisions to provide guidelines for design of steel-to-concrete connections.

6.2 Conclusions

The conclusions drawn from the results of this study are presented in the following subsections.

6.2.1 General. Regardless of the type of filler material used, the following conclusions were obtained :

1) All of the techniques tested (filling the annulus with epoxy or non-shrink grout, welding the nut to the steel element, use of spring washers with epoxy in the annulus), improved post-slip connection stiffness compared to a standard connection with no filler materials.

2) Specimens with a smaller hole clearance reached connection capacity at smaller interface slip.

3) An increase in clamping force beyond 60% of the yield capacity of the anchor bolt did not improve the behavior of connections with standard washers for a hole clearance of 3/16 in.

4) In general, ultimate connection strength was reduced by increasing the clamping force beyond 60% of yield in the anchor bolt (As much as 27% for specimens with standard washers and as much as 9% for spring-loaded connections).

5) Bearing failure of the concrete and anchor epoxy did not influence connection strength but affected connection stiffness.

6) Connection stiffness decreased progressively with repeated cycles of loading. The clamping force decreased during cyclic tests for connections with standard washers due to anchor bolt elongation and crushing of the concrete and anchor epoxy near the top of the anchor bolt embedment.

7) Stiffness degradation occurred faster with increased hole clearance.

6.2.2 Conclusions from tests incorporating non-shrink grout.

1) Sandblasting of the steel contact surface improved first-slip connection capacity by at least 22% and as much as 53%.

2) Load at first slip increased proportionally to the applied clamping force.

3) An increase in clamping force beyond 60% of the tensile yield capacity

of the anchor bolt decreased ultimate capacity and did not influence post-slip connection stiffness for a 3/16 in. hole clearance.

4) The use of a 1/2 in. thick grout interface layer reduced ultimate strength by approximately 15% but increased deformation capacity by a factor of two.

5) Cyclic loading had a detrimental effect on the stiffness of connections with non-shrink grout.

6) Cracking of the 1/4 in. interface layer did not appreciably influence connection stiffness and ultimate strength but increased deformation capacity by approximately 40%.

6.2.3 Conclusions from tests with epoxy-filled annulus.

1) Position of the anchor bolt in the hole did not significantly influence the behavior of connections with epoxy in the annulus.

2) Hand-tightened connections with an epoxy-filled annulus behaved similar to torqued connections with grout after first slip. However, the first-slip capacity of hand-tightened specimens may be too low to be of use in strengthening applications.

3) An applied bolt preload of 60% of the yield capacity of the anchor bolt significantly enhanced first-slip capacity if the anchor was straight. First-slip capacity was increased by at least 200% and 30% for connections with hole clearances of 3/16 in. and 7/16 in. respectively, as compared with hand-tightened connections.

4) Connections with an epoxy-filled annulus and a clamping force of 12 kips (60% of anchor bolt tensile yield capacity) were stiffer at all stages of loading than either hand-tightened connections with an epoxy-filled annulus or torqued connections incorporating non-shrink grout.

5) The improvement in stiffness with clamping force was less pronounced for specimens with a hole clearance of 7/16 in.

6) An initial five-degree bolt inclination reduced first-slip capacity by a factor of at least two. Post-slip stiffness was reduced if the bolt was inclined away from the direction of applied load.

6.2.4 Conclusions from tests with interface epoxy.

1) Interface epoxy increased first-slip connection capacity by a factor of at least two.

2) Sandblasting of the steel element (removal of the mill scale) increased first-slip connection capacity by a factor of 1.8 with respect to a companion test with an acetone-cleaned steel channel.

3) Elastic capacity increased proportionally to the applied clamping force.

4) The post-slip behavior obtained for epoxy-bonded specimens cannot be easily extrapolated to multiple-connection response.

6.2.5 Conclusions from tests with spring washers.

1) The hysteretic behavior of steel-to-concrete connections was enhanced by the use of spring-loaded washers.

2) Spring loaded connections had less stiffness after first slip than connections with standard washers. The difference was accentuated by increased hole clearance around the anchor bolt.

3) The first-slip capacity and energy absorption characteristics of the hystheresis loops for spring loaded connections was improved by increasing the clamping force beyond 60% of the yield strength of the anchor bolt (first-slip capacity increased by 21% in the average for both hole clearances studied). This was not observed for tests with standard washers.

4) The load-slip relationships obtained revealed that the clamping force was maintained during repeated cycles of loading at different stages.

5) The quality of epoxy filling the annulus was more critical when using conical washers since the material was not confined on the exterior surface of the steel element by the washer.

6.2.6 Conclusions from the welded nut test

1) The improvement in first-slip capacity and post-slip stiffness provided by welding the nut to the steel element was less significant than the improvement provided by the other fastening techniques implemented in combination with filler and interface materials.

2) The change in anchor bolt behavior introduced by fixing the nut to the steel element produced a decrease of approximately 13% in ultimate capacity as compared with the average from tests with an epoxy-filled annulus. Crushing of concrete surrounding the anchor bolt was more extensive for this detail.

6.3 Design Recommendations

1) The coefficient of friction proposed by Cook and Klingner was conservative for predicting first-slip capacity of tests with a non-shrink grout interface or epoxy-filled annulus. However, steel-to-concrete connections should still be designed for ultimate strength as bearing connections since the tension of the anchor bolt in service can not be accurately predicted. Recommendations included in Section 5.6 should be considered whenever first-slip capacity is important.

2) The ACI 349-85 shear friction provisions provided a safe estimate of the ultimate capacity of single anchor bolt connections like those tested. Increasing the clamping force beyond 60% of the anchor bolt tensile yield strength is not recommended if the ACI 349-85 provisions are used.

3) The use of a 1/2 in. thick non-shrink grout interface layer is recommended for connections where the anchor bolts are expected to undergo significant inelastic deformations, such as flexural strengthening of beam elements.

4) Different understrength factors should be assigned to connections depending on the application (bracing collector members, column jacketing, beam jacketing), predicted connection participation, and type of behavior expected. Additional reduction factors may be needed for long connections with large spacing between anchor bolts.

5) The quality of workmanship is of utmost importance for the performance of steel-to-concrete connections. Thorough field inspection by a qualified engineer should be performed. The designer should set ranges of tolerance for the initial inclination of the anchor bolts. If these tolerances are violated, the original design should be revised.

6.4 Further Research Needs

The following additional research is recommended in light of the results obtained in this study:

1) Investigate long-term preload relaxation of epoxy anchors,

2) Investigate the role of the following factors in affecting connection behavior:

a) Effect of initial bolt inclination greater than 5 degrees
b) Role of spring washers in reducing the effect of bolt inclination
c) Use of hardened washers in combination with oversize holes
d) Use of higher strength anchor bolts
e) Effect of low concrete strength (i.e. 2500 psi) on connection stiffness
f) Use of filler epoxy in combination with the welded nut detail

3) Investigate the distribution of forces to bolts in multiple-fastener connections to develop a simple and safe analytical model for design of steel-to-concrete connections. The validity of the analytical model(s) proposed should be confirmed by tests of long steel-to-concrete connections including selected variables from this study and/or from those suggested above. In addition, the effect of the following specific factors should be determined:

a) Joint length
b) Spacing between connectors
c) Relative proportions between the net area of the steel element and the total bolt area
d) Anchor bolt pattern (for instance two rows of connectors)

4) It is additionally recommended that the actual load-slip behavior of steel-to-

concrete connections be recorded in future testing of steel bracing or steel jacketing retrofit systems.

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VITA

Julio Jiménez-Pacheco was born in San José, Costa Rica, on September 6, 1968. The second son of María Marta Pacheco González and Eugenio Jiménez Bonilla. He received his High School Diploma from the Lycée Franco-Costaricien, Concepción de Tres Ríos, Costa Rica, in December 1984. He enrolled the School of Engineering at the Universidad de Costa Rica, San Pedro de Montes de Oca, Costa Rica, in 1985. He transferreed to The University of Texas at Austin in January 1988 where he obtained his Bachelor of Science in Civil Engineering Degree in May 1990. He enrolled the Graduate School of The University of Texas in September 1990.

Permanent address: Apartado 230
San José - 1000
Costa Rica

This thesis was typed by Julio Jiménez-Pacheco