An investigation of bond-slip behavior of reinforcing steel subjected to inelastic strains

by

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Longitudinal bar slip resulting from strain penetration at the end of flexural concrete members will result in a member end rotation and additional lateral deformations in members such as walls, columns and beams. The contribution of these member end rotations can account for as much as 35 percent of the total lateral deformations of these members. Therefore, the deformation resulting from strain penetration should be accurately accounted for when modeling reinforced concrete members subjected to flexural actions.

In order to model the bar slip as well as the associated member end rotation due to strain penetration of longitudinal bars adequately anchored into joints or footings, a local bond stress-slip constitutive relationship is typically required to be incorporated in detailed simulation models to model the interface between the reinforcement and concrete. However, the local bond-slip models available for analytical detailed simulation were developed from experimental tests conducted on reinforcing bars with short embedment length, where the slip of test bar occurred when they were subjected to strains well below the yield strain. Consequently, these models are strictly not applicable to critical flexural regions such as the plastic hinges experiencing significant nonlinear strains.

In order for the reinforced concrete structures to develop ductile response during moderate or severe earthquake excitations, these structures are designed to develop plastic hinges in the critical moment regions at the wall and column bases as well as at the beam end regions. In these situations, the longitudinal reinforcement at the connection interface experiences as high as 25 times the yield strain, causing the rebar to slip over the entire (e.g., beam ends) or partial length (e.g., column and wall bases) through deterioration of local bond at the steel-concrete interface. Given that the objective of detailed analysis should be to
produce satisfactory global and local responses, it should be realized that accurately representing the local bond-slip behavior of longitudinal reinforcing bar experiencing inelastic strains is critical. This measure will enable the bar slip and member end rotations due to strain penetration to be quantified accurately.

An alternative approach that maybe suitable for fiber-based analysis is to model the bar stress vs. slip hysteretic response directly as proposed by Zhao and Sritharan (2007), thereby capturing the local and global responses accurately. Their model was based on limited test data derived from pull out tests conducted on reinforcing bars with long embedment length, forcing the bars experienced large inelastic strains.

In consideration of the state-of-the-art summary presented above on the bond-slip behavior of reinforcing steel subjected to inelastic strains, an experimental investigation was designed and completed recently. In these tests, test bars were designed with sufficient anchorage lengths as would be the case for bars anchored into foundations. A total of five bars of two different bar sizes (i.e., #6 and #8) were tested under both monotonic and cyclic loadings. Following collection of quality data from these tests, the model proposed by Zhao and Sritharan was examined and found to be appropriate for modeling the bar stress vs. loaded-end slip relationship at the interface between column or wall and the foundation.

Through an analytical investigation combined with measured data along the embedded portion of the bar length, it was further found that the bond strength reduces as the reinforcing bar experienced inelastic strains. Using the suggestion of Wang (2008) that this reduction could be accounted for through a modification factor, an investigation was conducted by comparing the predicted bar stress vs. loaded-end slip relationship derived from this analytical investigation to the pullout test results. It was found that the
modification factor proposed by Wang (2008) was useful in improving the global response. In this analysis process, the strain and local slip distributions along the bar embedment length were examined for when the bar was subjected to strains well above the yield strain. Significant local slip was found to occur along the embedment length over the portion of the rebar experiencing significant inelastic strains, which was consistent with the measured data.

Based on the completed study it is concluded that: 1) local bond-slip relation will be different for a reinforcing bar subjected inelastic strains than those found from bars subjected to elastic strains; 2) the existing local bond models may be modified with a factor such as that proposed by Wang (2008) to account for the effects of inelastic strains; and 3) strain penetration model widely of Zhao and Sritharan that is widely used in fiber-based analysis sufficient will sufficient captures the effects of strain penetration effects.
CHAPTER 1. INTRODUCTION

This chapter presents an overview of the problems addressed in this project, the research scopes and objectives, as well as a discussion of the significance/benefits of this research. The final section of this chapter forecasts the organization of the thesis.

1.1 Overview

Over the past few decades, numerous earthquakes have occurred around the world, several of which resulted in significant structural damage which was associated with a great amount of financial loss, and more importantly, human loss. The February 2011 Christchurch earthquake severely damaged one of the largest cities in New Zealand, which resulted in 185 fatalities and a total estimate damage of NZ$20-30 billion. To reduce the earthquake risk, improved understanding of structural earthquake behavior, advancement of the methods used to predicting structural response, and the development of new seismic design procedures will be required. Significant research effort has been devoted to the development of behavioral models and modeling tools to predict the earthquake response of reinforced concrete structure with a high level of accuracy.

Zhao and Sritharan (2007) noted that in flexural concrete members, strain penetration occurs along longitudinal reinforcing bars that are fully anchored into connecting concrete members (e.g., bridge joints or footings), causing bar slips along a partial anchorage length and thus end rotations to the flexural members at the connection interfaces. These longitudinal bar slips resulting from strain penetration at the end of flexural concrete members will result in member end rotations. The contribution of these member end rotations to total lateral deformations in members such as walls, columns and beams could be relatively large. Previous experimental research indicated that the longitudinal bar slips resulting from strain penetration at the end of flexural concrete members and associated member end rotations could account for as much as 35 percent of the total lateral deformations of flexural members [Calderone et al., (2000), Kowalsky et al., (1999), and Saatcioglu et al., (1992)]. Therefore, the deformation resulting from strain penetration should be accurately accounted for when modeling reinforced concrete members subjected to flexural actions.
Neglecting the longitudinal bar slips resulting from strain penetration in reinforced concrete structural analysis will underestimate the lateral deformations and member elongations, while overestimating the member stiffness, hysteretic energy dissipation capacities, steel strains, and section curvatures. In order to model the bar slip as well as the associated member end rotation due to strain penetration of longitudinal bars fully anchored into joints or footings, a local bond stress-slip constitutive relationship is typically required to be incorporated in high-resolution finite element modeling of reinforced concrete structures subjected to linear or nonlinear loading, to model the bond zone between the reinforcement and concrete. The accuracy of the detailed numerical simulation models of such structural systems therefore depends on the accuracy of the local bond stress-slip constitutive relationship. However, the local bond-slip models available up to now for analytical detailed simulation were developed from experimental tests conducted on reinforcing bars with short embedment length, where the bond stress was assumed to be uniformly distributed along this short embedment length and the slip of test bar occurred when they were subjected to strains that were well below the yield strain. One of the most widely used and well-recognized local bond-slip models, proposed by Eligehausen, was based on pullout tests conducted on reinforcing bars with a short embedment length (i.e., 5 \(d_b\), where \(d_b\) is the bar diameter) at the University of California, Berkeley. Consequently, these models are strictly not applicable to critical flexural regions such as the plastic hinges experiencing significant nonlinear strains when subjected to moderate or severe earthquake excitations.

In order for the reinforced concrete structures to develop ductile response during moderate or severe earthquake excitations, these structures are designed to develop plastic hinges in the critical moment regions at the wall and column bases as well as the beam end regions. In these situations, the longitudinal reinforcement at the connection interface experiences as high as 25 times the yield strain, causing the rebar to slip over the entire (e.g., beam ends) or partial length (e.g., column and wall bases) through deterioration of local bond at the steel-concrete interface. Given that the objective of detailed analysis should be to produce satisfactory global and local responses, it should be realized that accurately representing the local bond-slip behavior of longitudinal reinforcing bar experiencing inelastic strains is critical.
An alternative approach that maybe suitable for fiber-based analysis is to model the bar stress vs. slip hysteretic response directly as proposed by Zhao and Sritharan (2007), thereby capturing the local and global responses accurately. Their model was based on limited test data derived from pull out tests conducted on reinforcing bars with long embedment length, forcing the bars to experience large inelastic strains. In consideration of the state-of-the-art summary presented, an experimental investigation which could capture the bond-slip behavior of a longitudinal reinforcing bar adequately anchored in a well-designed reinforced concrete member, as would be the case for bars anchored into foundations, is required to be designed and conducted.

Previous researchers [Shima et al. (1987), Mayer and Eligehausen (1998), Lowes et al., (2004), and etc] noticed that the yielding of reinforcing bar could reduce the bond strength significantly due to the reduced contact area between the reinforcing bar and surrounding concrete. Therefore, the local bond stress-slip relationship may change significantly when the reinforcing bar yields and a local bond-slip model considering inelastic strains is required to obtain a satisfactory local and global response of reinforced concrete members. Lowes (2004) and Wang (2008) suggested that this bond strength reduction due to inelastic strains could be accounted for through a modification factor. Under this condition, a modification factor was proposed by Wang (2008) based on back calculation from global response. Wang (2008) proved that this modification factor was useful in improving the bar force vs. loaded-end slip relationship.

1.2 Research Scope and Objectives

The overall goal of the research conducted here is to study the bond-slip behavior of reinforcing steel subjected to inelastic strains. All of the testing for this thesis was performed at the Iowa State University Structural Laboratory. The thesis will focus on four main objectives: (1) conducting pullout tests to accumulate the bar stress vs. loaded-end slip relationship for a deformed bar anchored in a well-confined concrete block with sufficient embedment length, (2) providing test data to examine the accuracy of the hysteretic bar stress vs. loaded-end slip model proposed by Zhao & Sritharan (2007), (3) examining the strain distribution and local slip distribution along the embedment length of a reinforcing bar anchored in a well-confined concrete block when the reinforcing bar is subjected to strains
well above the yield strain by taking the modification factor proposed by Wang (2008) as appropriate, and (4) developing a local bond-slip model considering inelastic strains.

The first objective is obtained by testing deformed bar adequately anchored in a well-confined concrete block straightly with sufficient embedment length as would be the case for reinforcing bar anchored into foundations. The strain penetration effect was satisfactorily represented, where large inelastic strains were developed at the column-footing interface and the test bar was pulled out with a relatively large ultimate loaded-end slip. The embedment length was sufficient allowing the test bar to be fully developed up to the ultimate strength. The bar stress and loaded-end slip were measured directly during the test. A total of five bars of two different bar sizes (i.e., #6 and #8) were tested under both monotonic and cyclic loadings. The test bar was loaded in one end and the other end was set to be free.

The hysteretic bar stress vs. slip model proposed by Zhao and Sritharan was examined through comparisons to the measured bar stress vs. loaded-end slip response in the pullout tests and the second objective is obtained.

As previously discussed, the modification factor proposed by Wang (2008) was proved to be able to improve the bar stress vs. loaded-end slip response. This modification factor was applied to the well-recognized local bond-slip model proposed by Eligehausen (1983) as a multiplier of the local bond stress to account for the local bond stress reduction due to inelastic strains. The improved local bond-slip model was incorporated into an analytical model (to be discussed in detail in the analytical study chapter) to simulate the bar stress vs. loaded-end slip relationship. This modification factor was examined by comparing the simulated bar stress vs. loaded-end slip relationship developed from the analytical model to the pullout test results. In this analysis process, the strain and local slip distributions along the bar embedment length were examined for when the bar was subjected to strains well above the yield strain.

1.3 Research Significance

The experimental and analytical studies conducted in this thesis will advance the understanding of the bond-slip behavior of reinforcing bars subjected to inelastic strains, so as to improve the analysis capabilities. The strain penetration effect was simulated in the experimental study and satisfactory set of test data was collected in the pullout tests.
Following collection of quality data from these tests, the model proposed by Zhao and Sritharan was examined and found to be appropriate for modeling the bar stress vs. loaded-end slip relationship at the interface between column or wall and the foundation. In addition, the experimental study gave a direct local slip measurement by using the special instrumentation instead of doing integration of strains along the embedment length of the deformed bar.

The analytical study proved that the modification factor proposed by Wang (2008) was useful in improving the global response represented by a relationship between bar stress and loaded-end slip by comparing the predicted response derived from this analytical study to the pullout test results. In this analysis process, the strain and local slip distributions along the bar embedment length were examined for when the bar was subjected to strains well above the yield strain. Significant local slip was found to occur along the embedment length over the portion of the rebar experiencing significant inelastic strains, which was consistent with the measured data.

1.4 Thesis Organization

The thesis consists of six chapters. Chapter 1 introduces the subject and states the objectives of the research. Chapter 2 is literature reviews of past research conducted on bond behavior of deformed reinforcing bar anchored in concrete block. The procedure and test setup of pullout tests are described in Chapter 3. Chapter 4 presents the results of the testing as well as analysis and discussion of the test results. An analytical study is designed and conducted on specimens with long embedment length in Chapter 5. Finally, Chapter 6 presents a summary of the research, conclusions and recommendations for future research.
CHAPTER 2. LITERATURE REVIEW

This chapter consists of four parts: (1) a brief description on bond behavior of deformed bars, (2) a discussion of bond in actual reinforced concrete structures, (3) a review of the literature that summarizes major research and contributions to the field, and (4) an analysis of gaps in existing research that reflects how this project contributes new knowledge to the field.

2.1 Bond Behavior of Deformed Bars

For the optimal design of reinforced concrete structures, the efficient and reliable force transfer between reinforcement and concrete is of key importance. Three mechanisms that transfer forces from deformed bars to surrounding concrete as illustrated in Figure 1 [ACI 408R-03] are as follows:

- Chemical adhesion between deformed bar and concrete;
- Frictional forces arising from roughness of interface, forces transverse to bar surface, and relative slip between deformed bar and surrounding concrete;
- Mechanical anchorage or bearing of ribs against concrete surface.

![Figure 1: Force transfer mechanisms [ACI 408R-03]](image)

This force transfer from deformed bar to surrounding concrete is referred to as bond. After the initial slip of the bar, chemical adhesion is lost and most of the force is transferred through friction and bearing. The bearing forces of the ribs against concrete result in two components: one parallel to the deformed bars providing bond resistance, and the other perpendicular to the deformed bars that may cause transverse cracks. Correspondingly, there are two typical types of bond failure: splitting failure and pullout failure. If either the
concrete cover, bar spacing, or transverse reinforcement is not sufficient, transverse cracks will form easily under flexural bending and these transverse cracks may propagate through the entire concrete cover, leading to a splitting failure (Figure 2 (a)). In this case, the maximum bond stress depends on the maximum tensile stress provided by the surrounding concrete and the main force transfer mechanism is the rib bearing. If the confinement is sufficient to restrain the transverse cracks propagation and thus the splitting failure is prevented, the system may fail by shearing along a surface at the top of the ribs around the bars, resulting in a pullout failure (Figure 2 (b)) [ACI 408R-03]. In this case, the force transfer mechanism changes from the rib bearing to friction before a pullout failure occurs. A pullout failure occurs when the integration of bond stress along the bar is insufficient to resist the external applied load. It is essential to realize that the reinforcing bars embedded in concrete may not fail decisively with pullout or splitting. In real structures, a combined failure may occur, depending on the details of the reinforced concrete members and loading conditions.

![Figure 2: (a) End view of a member showing splitting cracks between bars and through the concrete cover; (b) Side view of a member showing shear crack and/or local concrete crushing due to bar pullout [ACI 408R-03]](image)

### 2.2 Bonds in Reinforced Concrete Structures

In the analysis of reinforced concrete structures, the bond between reinforcement and surrounding concrete is often considered through a bond stress-slip relationship. Bond stress is typically defined as the equivalent unit shear stress acting parallel to the reinforcing bar at the interface between reinforcing bar and concrete. Slip is defined as the relative displacement of reinforcing bar with respect to concrete. The bond stress-slip relationship expresses the local bond stress as a function of the local slip at any location along a bar. This
relationship is also used in detailed analytical simulation models to predict the force-end behavior of anchored bars.

Stiffness, ductility, and energy dissipation are three characteristics that control the behavior of reinforced concrete structures. Bond-slip for bars anchored within a connection strongly influences these three characteristics. Therefore, the bond between reinforcement and surrounding concrete controls the behavior of reinforced concrete structures. A typical 3-story reinforced concrete frame subjected to a lateral load (P) is introduced, in order to analyze the bond stress and relative slip in actual reinforced concrete members (Figure 3). There are four different cases to be discussed, which generally cover all the stress states and slip conditions present in a reinforced concrete structure.

![Figure 3: Four bond cases in a reinforced concrete frame subjected to a lateral load](image)

### 2.2.1 Case 1 and Case 2: beam and column subjected to flexure

Concrete in flexural members (e.g., beams or columns subjected to flexural action) will crack, which causes the bond stress to vary along reinforcing bars. For region 1 (Figure 4), the beam segment is mainly subjected to flexure. The steel tension force along the length of
a cracked concrete member is modified by the bond between the steel and its surrounding concrete in such a way that its value varies from a maximum across each crack to a minimum at a point about halfway between adjacent cracks. The bond stress and slip condition of columns subjected to flexure (Figure 5) is similar to that of region 2 (Figure 4) for beams. The slip is usually small in these two cases due to the flexural cracks, but the bond stress is relatively large.

![Figure 4: Bond stress and slip condition of beams subjected to flexural action](image)

![Figure 5: Bond stress and slip condition of columns subjected to flexural action](image)

### 2.2.2 Case 3: beam-column joint

The plastic hinges are designed to locate at the beam ends (as shown by the red shaded area in Figure 6) for the reinforced concrete frame to develop ductile response, where the beam longitudinal bars could experience slip due to the strain penetration during moderate or severe earthquake excitations. In an interior joint, the force of beam longitudinal bars passing continuously through the joint changes from compression to tension due to the lateral load (Figure 6). This changing force causes a push-pull effect requiring high bond strength.
and an adequate development length in order to fully develop the reinforcing bar within the joint. Beam longitudinal bars are anchored inside beam-column joints, where sufficient development length is usually not provided due to the limitation of column width. Therefore, under severe earthquake load, slip between reinforcing bars relative to surrounding concrete would occur along the entire length of the beam bar within the joint, leading to rotations that can account for significant lateral deformation. Large bond stress and large slip are expected to occur at the joint interface in this case.

![Figure 6: Bond stress and slip condition of beam column joints](image)

**2.2.3 Case 4: column-footing connection**

The plastic hinges are designed to locate at the column ends (as shown by the red shaded area in Figure 7) for the reinforced concrete frame to develop ductile response, where the column longitudinal bars could experience slip due to the strain penetration during moderate or severe earthquake excitations. Within the column-footing connections, the embedment length is usually adequate for fully developing the reinforcing bar. In addition, the longitudinal bars anchored into footings are often detailed with 90° hooks at the end to improve constructability and reduce the depth of footings. However, the accumulated elongation of the reinforcing bar at the column-footing interface can cause large column-end rotations, which significantly contribute to the lateral deformation of reinforced concrete columns and structures under severe earthquake excitations. Unlike the beam bars anchored into interior beam-column joints, the slip experienced by the longitudinal bars anchored into footings may occur along only a portion of the anchorage length. Large bond stress and large slip occur at the connection interface as shown in Region 3 (Figure 7). Small bond stress and small slip occur along the embedment length into the foundation as shown in Region 4.
Figure 7. Bond stress and slip condition of column foundation connections

Based on the previous discussions, it is noticed that bond in an actual reinforced concrete structure generally involves two scenarios: bond in flexural condition (Case 1 and Case 2) and bond in anchorage condition (Case 3 and Case 4). Previous experimental tests indicated that bond damage in anchorage regions (such as the beam-column joint or the column-foundation connection) significantly contributed to the loss of stiffness and strength in entire reinforced concrete structures [Lowes et al., 1999]. Therefore, understanding the bond behavior in anchorage regions is significantly important. In addition, nonlinear deformation is expected to occur at the plastic hinges in the reinforced concrete structures subjected to moderate and severe earthquake excitations, where the longitudinal bar at the beam-column joints or within the column-footing connections could experience large inelastic strains, so the bond behavior in anchorage condition with emphasis on inelastic strains is therefore the subject of the research reported herein.

2.3 Experimental Study of Bond Behavior of Deformed Bars

Two types of tests used to study bond behavior include: pullout tests and beam tests. The research presented in this thesis focuses on pullout tests because pullout tests clearly...
represent the bond condition in anchorage zones. Beam tests provide more realistic measures of bond strength in the tension zone of concrete beams (i.e., beams subjected to flexure), because both the reinforcing bar and concrete is placed in tension and the dowel action is also represented. However, beam tests are typically used to determine splice anchorage, which is beyond the scope of this research.

Over the past several decades, numerous experiments have been conducted to study the local bond stress-slip relationship of a deformed bar anchored in concrete and several local bond stress-slip models have been proposed. Representative works are reviewed and summarized in the following sections including a brief description of test procedures, test results and derived bond-slip models. In these local bond-slip models, the local bond stress is represented by \( \tau \), the slip is \( s \), the bar diameter is \( d_b \), the concrete compressive strength is \( f'c \) and the steel yield strength is \( f_y \). Other parameters pertained to certain models are defined following the model description.

### 2.3.1 Double pullout test

The double pullout test has been used to determine local *average* bond stress-slip relationship of bond in flexural members (Case 1 and Case 2). In this case, the reinforcing bar is encased concentrically in a long rectangular concrete prism or cylinder. Tensile forces are applied at both ends of the reinforcing bar. The slip, where the slope of strain distribution curve approaches to zero, is used as the reference point (the reference point is usually set at the middle point of the specimen due to symmetry for simplified consideration) [Shima et al, 1987]. Tensile splitting cracks tend to readily form under the radial component of the rib-bearing forces against concrete due to flexural bending. Once these transverse tensile cracks propagate through the entire concrete cover, a splitting failure would occur. The specimen in the double pullout test was used to simulate the bond-slip condition in the tension zone of a concrete beam between primary flexural cracks, such as the central tensile part of a simple supported beam in bending. With this type of setup, it is possible to measure the steel strain variation along the embedment length and slips at both ends, from which the bond stress and local slip could be derived at any position along the embedment length and a local bond stress-slip relationship can be obtained. Due to potential flexural cracks in the beams, the slip is usually small.
There are two opposite viewpoints regarding the bond stress-slip relationship of deformed bars in double pullout test: position-dependent and position-independent (i.e., the local bond stress–slip relationship depends on the location along the embedment length or not). Nilson (1972) conducted double pullout tests with a specimen reinforced with special internally instrumented #8 deformed bars (Figure 8). The reinforcing bar strains ($\varepsilon_s$) were measured with internally installed strain gages and the concrete strains ($\varepsilon_c$) were measured by embedded concrete strain gages. In this model, both bond stress and slip were established as a continuous function of distance and the bond-slip model was represented as:

$$\tau = 3100 \cdot (1.43x + 1.5)f'_c$$  \hspace{1cm} (2.1)

where $x$ is the distance from the loaded-end in inches, and $f'_c$ is the concrete compressive strength is in psi. However, Nilson (1972) did not arrive at a general conclusion, because only on bar size (i.e., #8) and one type of specimen was tested and only a limited range of concrete compression strength was used.

![Figure 8: Specimen of double pullout test conducted by Nilson (1972)](image)

Mirza and Houde (1979) conducted thirteen double pullout tests with specimens similar to Nilson’s specimens. The steel stress was kept below the yield steel stress. Based on the test data, a local bond stress-slip model that was independent of position was derived. The bond stress and local slips near the loaded end were used to obtain the following local bond stress-slip model:

$$\tau = 1.95 \cdot 10^6 s - 2.35 \cdot 10^9 s^2 + 1.39 \cdot 10^{12} s^3 - 0.33 \cdot 10^{15} s^4$$  \hspace{1cm} (2.2)

where the slip $s$ is measured in in. and the bond stress is measured in psi. It was found that the position-independent characteristics of the local bond stress-slip relationship applied to
the curve before reaching the peak bond stress value (at an average of 0.0012 inches slip). After passing the peak point, the relationship was again found to be position-dependent. The bond stress after the peak point could be constant or decrease with a further increase in slip. Their work indicated that a general bond-slip relationship that is independent of position may exist.

Kankam (1997) conducted experimental tests to establish the relationship among bond stress, steel stress, and slip in reinforced concrete structures by using double pullout test with specimens reinforced with internal instrumented plain bars and deformed bars. The relationship among bond stress, steel stress and slip was derived from the steel strain distribution along the embedment length and was represented by empirical formulas. It was concluded that the reinforcing bar stress affected the local bond-slip relationship of plain bars, which revealed that the position-dependent characteristics of the local bond stress-slip relationship observed earlier by Nilson (1972), and Mirza & Houde (1979) could be a result of the different steel stress levels in reinforcing bars. Tensile stresses in bars would reduce the bond stress, while compressive stresses would increase the bond stress. However, this conclusion did not apply to deformed bars. The proposed relationship for deformed bars was a function of the position relative to the middle point:

$$\tau = (35 - 0.3x)(s)^{0.5}$$

(2.3)

where x is the distance from the mid-point of the specimen in mm.

Study on double pull-out tests provided a general idea of the local bond-slip model. A general position-independent local bond-slip model may exist. Double pullout tests are limited because the bond stress and slip have to be determined indirectly from measured strains along the embedment length. Concrete cracking and steel stresses affect the strain measurement and thus the bond behavior. Therefore, most models were not formulated following a general format. Pullout tests with short anchorage lengths have been used to obtain isolated bond-slip characteristics.

2.3.2 Single pullout test

Single pullout test has been used to determine the local bond stress-slip relationship of bond in anchorage regions (Case 3 and Case 4). In this case, the reinforcing bar was
typically loaded at one end and the other end was set to be free, which resulted in zero strain at the free end. Adequate transverse reinforcements was typically placed that the yielding as well as failure were dominated by pullout of the bar. The specimen was used to simulate the confined region at the beam-column joints or column-footing connections. Based on the embedment length, the single pullout test was subdivided as single pullout test with short embedment length and single pullout test with long embedment length.

2.3.2.1 Single pullout test with long embedment length

Earlier single pullout tests (1970s) were conducted on reinforcing bars with long embedment lengths and the reinforcing bars were typically strained before reaching the yield capacity. The global response represented by a relationship between the applied tensile force and the slip at the loaded end was developed. Local bond stress and slip were determined similarly to the double pullout test, with the exception of the slip at the free end being used as reference point instead of a middle point. Like the double pullout test, the single pullout test was also limited because the bond stress and slip had to be determined indirectly from the measured strains along the embedment length. Concrete cracking and steel stresses in tension influenced the strain measurement; therefore, the derived model depended greatly on the strain measurements in each test. The accuracy of the steel stress-strain model also had an effect on the bond stress calculation because the bond stress was calculated based on the difference of bar stress according to force equilibrium. Relatively large slip (compared to the slip measured at the double pullout tests) at the loaded-end was expected from this kind of test.

A study presented by Viwathanatepa et al. (1979) was one of the first investigations on the force vs. loaded-end slip response of anchored deformed reinforcement. Seventeen specimens of single bar embedded in a well-confined concrete block represented longitudinal beam reinforcement anchored in a well-confined interior beam-column joint. These seventeen specimens were subjected to three kinds of loading: (1) pull-only loading; (2) push-simultaneously pull-loading (represented the push-pull effect experienced by the longitudinal bar in an interior beam-column joint) and (3) cyclic loading. Figure 9 shows the typical bar stress vs. displacement relationship of pull only test subjected to monotonic loading and Figure 10 presents the local bond stress-slip histories at points along the
embedded length of the bar for specimen with #8 reinforcing bar and a 25 in. embedment length. The local bond stress was computed from the steel strains measured at two adjacent locations along the bar and also the experimentally observed monotonic steel stress-strain relationship. The specimen was subjected to a monotonic pullout under displacement control at one end only. Two elastic cycles were performed prior to imposing the final monotonic loading to failure. It was shown that the bond-slip response subjected to monotonically increasing load was initially relatively stiff with stiffness reducing as the peak bond capacity was approached. Once bond capacity was reached, increased slip demand resulted in reduced capacity until the minimum bond capacity was maintained. The bar stress vs. loaded-end slip relationship, strain distribution along the reinforcing bar, bond stress distribution along the reinforcing bar and local displacement distribution were investigated extensively under the three loading conditions. Test results for specimens with nominal bar sizes ranging from No. 6 to No. 10 indicated that bond capacity decreases slightly as bar size increases. However, this study was conducted based on the assumption that the main embedded steel had to remain in the elastic range. Therefore, Viwathanatepa et al suggested that future study incorporating the yielding of the rebar is essential because the major degradation in bond occurs at and after yielding of the rebar.

Figure 9: Typical bar stress vs. displacement diagram of pull only test subjected to monotonic loading [Viwathanatepa et al. (1979)]
Figure 10: Local bond stress vs. slip relationship for #8 reinforcing bar subjected to tension and compression loading at opposite ends of the bar [Viwathanatepa et al. (1979)]

Ueda et al. (1986) developed a computer program to predict the force vs. loaded-end displacement characteristics for a beam bar anchored within an exterior column-beam connection subjected to large inelastic loading. The program incorporated a local bond stress-slip relationship derived from tests of local bond specimens by Hawkins et al. (1982), the experimental observed stress-strain properties for the bar, and the force equilibrium as well as the compatibility equations between steel and concrete. The local bond stress-slip model included the influenced parameters into the model, such as the concrete compressive strength, bar size, bar surface geometry, and load history. Ueda et al. found that strain-hardening characteristics of the bar also played an important role in the force vs. loaded-end displacement relationship for a bar and the required development length for a bar, besides factors specified in Chapter 12 of ACI318-08. Twenty two specimens representing the ideal exterior beam-column connections were tested by the researchers to validate the computer program. The test variables included the concrete compressive strength, bar size, the yield strength of the reinforcing bar, anchorage type and embedment length. Figure 11 shows the comparison of measured and predicted load vs. loaded-end displacement curve for specimen with #6 reinforcing bar and an anchorage length of 24 in. The step change in the elastic portion of the predicted curve corresponded to pullout of a cone of unconfined concrete at the
loaded end of the bar. Ueda et al. concluded that one of the reasons for a difference between the measured and predicted results was not accounting for the bar area reduction caused by gross yielding associated with the Poisson’s effect in the local bond-slip model. The local bond-slip relationship used in this analytical model was therefore concluded to be more applicable for non-yielding bar only. The analytical model was also used to investigate the effects of concrete strength, bar size, bar stress-strain characteristics and bar embedment length on the load-deformation relationship. The effect of bar size and bar yield strength on the load-deformation relationship was illustrated in the Figure 12 and Figure 13 respectively. Because the yield load depended on both the bar size and the yield strength, the ordinates of the vertical axis in these two figures were normalized and expressed as the ratio of the applied load to the yield load. It showed that the ratio of the force to the yield force corresponding to a given slip increased with decrease of the bar size and increased with the decrease of the bar yield strength. Therefore, Ueda et al concluded that the bond resistance for a given slip increased with decrease in the bar size and bar yield strength.

Figure 11: Comparison of measured and predicted load-displacement relationship

[Ueda et al. (1986)]
Figure 12: Effect of bar size on predicted force-loaded end displacement relationship [Ueda et al. (1986)]

Figure 13: Effect of bar yield strength on predicted force-loaded end displacement relationship [Ueda et al. (1986)]
2.3.2.2 Single pullout test with short embedment length

A single pullout test with a short embedment length is favored by several researchers and has been widely used in recent experiments so as to achieve a relatively uniform distributed bond stress along the embedment length in order to interpret the characteristics of bond behavior. By assuming an evenly distributed bond stress along the short embedment length, bond stress is calculated by dividing the applied load by the contact area (rebar perimeter multiplied by the embedded length) and the slip is usually obtained by measurements taken at either the loaded-end or free end. Therefore, a bond-slip model can be derived directly based on the test data.

Various types of pullout tests with short embedment lengths have been carried out to evaluate the local bond stress-slip model. Typical in these tests, the yield capacity of the deformed bar was far in excess of the total bond resistance that could be obtained within this short embedment length. Therefore, the deformed bar was pulled out before it reached the yield strength. The local bond-slip relationship obtained in these tests represents an upper limit for the bond resistance in elastic range.

Eligehausen et al (1983) developed one of the most widely used and commonly recognized bond-slip models based on single pullout test with short embedment length. Within the last 20 years, various refined computer models were proposed to implement this model, and a few similar models were developed to represent bond-slip deformations in reinforced concrete members under monotonic loads.

Eligehausen et al. (1983) tested 125 pull-out specimens to predict the local bond stress-slip relationship of deformed reinforcing bars embedded in concrete that were subjected to generalized excitations. The specimens were designed to simulate a beam reinforcing bar embedded in an interior beam-column joint (Figure 14). Primary parameters that were investigated in this study included: confining reinforcement, bar size and deformation pattern, concrete compressive strength, clear bar spacing, transverse pressure, and loading history. A single deformed reinforcing bar with an anchored length of five times the bar diameter (i.e., 5d_b, where d_b is the bar diameter) in a concrete block was used in the tests to reduce the potential of non-uniform bond stress distribution, while large enough to prevent scatter of test results. All the test specimens with confined concrete failed as a result of
pullout failure at steel stress below the yield strength. The specimens were tested under displacement control by applying load at one bar end. The bond stress was calculated and the slip was measured as the movement of the free end of the bar with respect to the concrete block. This study contributed to the development of a general shape of local bond-slip relationship (Figure 15). Also, the specimen used in this test and the developed model are widely used for analytical investigation involving bond-slip models.

Figure 14: Eligehausen’s test specimen [Eligehausen et al. (1983)]
Five of the observations in Eleighausen et al. (1983) that have a bearing on the research reported in this thesis are:

- An upper limit for an effective restraining reinforcement exists for the effect of the area of vertical bars beyond which the bond behavior cannot be improved further;
- The maximum bond resistance decreases slightly as bar diameter increased;
- Under monotonic loading, the maximum bond strength increased almost proportionally to $\sqrt{f'c}$, while the slip at the maximum bond strength decreased almost proportionally to $\sqrt{f'c}$;
- The maximum bond resistance and the ultimate frictional resistance were increased with the transverse pressure; and
- The slip at maximum bond resistance increases slightly with the increase of transverse pressure.

In addition, five controlling parameters that were determined from the pull-out test data:

- The peak bond stress, expressed as: $\tau_1 = 31\sqrt{f'c}$ (psi);
- The frictional bond resistance, expressed as: $\tau_3 = 0.35\tau_1$ (psi);
- $s_1 = 0.15c_0$;
- $s_2 = 0.35c_0$; and

Figure 15: Local bond stress-slip model proposed by Eligehausen et al. (1983)
\[ s_3 = c_0, \text{ where } c_0 \text{ is clear lug distance.} \]

This local bond-slip relation proposed by Eligehausen et al. has been adopted by CEB-FIP Model Code 1990 [CEB-FIP (1993)] for reinforcing bars anchored in a concrete block with enough concrete cover \((c)\) \((i.e., c/d_b \geq 5)\) and good anchorage condition. The following parameters are suggested by MC90, \(s_1 = 1 \text{ mm}, s_2 = 3 \text{ mm}, s_3 = \text{ clear rib spacing, } \tau_1 = 2.5 f_{ck}^{1/2}, \)
\[ \tau_3 = f_{ck}^{1/2}, \text{ and } \alpha = 0.4, \text{ where } f_{ck} \text{ is the characteristic concrete compressive strength.} \]

Hawkins and his coworkers (1982) tested 30 specimens simulating beam bars anchored in beam-column joints for seismic loading. The test was conducted at elastic range by assuming that none of the bars were stressed beyond the yield strength during the tests. The effect of bar diameter, concrete strength and loading history were investigated. The embedded length varied from one lug spacing \((i.e., 0.68 d_b)\) to four lugs spacing \((i.e., 2.72 d_b)\). It was observed that the results varied widely for similar specimens with one lug bonded length \((i.e., 0.68 d_b)\), while remained consistent with 4 lugs bonded length \((i.e., 2.72 d_b)\). Also, higher peak bond strength occurred if tests with two-lug \((i.e., 1.36 d_b)\) spacing were conducted, compared to one or four-lug spacing. This study reminded the later researchers to select a proper embedded length for the test.

The maximum local bond strength was nearly proportional to the compressive strength of concrete up to 5 ksi. As the concrete compressive strength passed 5 ksi, the maximum bond stress remained unchanged, while the ductility still increased. By using data collected from a specimen with a 2 lug-bonded length \((i.e., 1.36 d_b)\), approximately as much as two times maximum local bond stress was observed in Hawkins’ model \((4500 \text{ psi})\) compared to that of the model developed by Eligehausen \((2200 \text{ psi})\). The higher bond strength may have resulted from the short embedment length used in Hawkins’s study. The post-peak bond stress decreased almost linearly with increasing slips, which was in agreement with the conclusion derived by Eligehausen. Hawkins et al. concluded that the size and geometry of the bar as well as the concrete compressive strength affected the bond strength and the overall bond-slip behavior. The model (Figure 16) was developed based on the test data of specimens with 2 lugs \((1.36 d_b)\) bonded length.
Figure 16: Local bond stress-slip model proposed by Hawkins et al. (1982)

The response can be divided into three linear stages (Figure 16): (Stage 1) the un-cracked response (the stiffness of the local bond stress vs. slip relationship is proportional to the square root of the concrete compressive strength and is independent of bar size); (Stage 2) the internal cracked response, and (Stage 3) the sliding shear response. The controlling parameters for the model were determined by statistical analysis of the test data and are expressed as follows:

**Stage 1:**

\[ K_1 = 17 \sqrt{f'_c} \ (MPa) \]

\[ S_c = \left( \frac{\tau_{\text{max}} - K_2 S_0}{K_1 - K_2} \right) \]

**Stage 2:**

\[ K_2 = 950 \times \left( \frac{f'_c}{1000} \right)^{2/3} + 90 - \frac{452000}{d_b^3} \]

\[ S_0 = 8 \times \left( \frac{f'_c}{1000d_b} \right)^{1/3} + 310 \frac{d_b}{f'_c} \]

\[ \tau_{\text{max}} = 190 \times \left( \frac{f'_c - 16}{300} \right)^{2/3} + \frac{1.25 \times (32800 - d_b^3)}{f'_c d_b^{3/2}} \leq 34.5 \]
Stage 3:  \[ K_3 = \frac{-6 f_{c'}^*}{1000} + 4 - \frac{249000}{d_p^3} \]

\[ S_n = 0.5 \times (S_{lug} + W) \]

where, \( S_{lug} \) and \( W \) are the lug spacing and lug width, respectively.

The research conducted by Hawkins et al. showed that the shape of the monotonic bond-slip curve was related to the bar deformation pattern. The deformation pattern of reinforcing bar was characterized by lug spacing \( l_s \), lug height \( l_h \), and the lug face angle \( \theta \) as defined in Figure 17. Pochanart and Harmon (1989) used reinforcing bars machined from 1 in. diameter steel rods to obtain varying lug height, lug spacing and lug face angle, in order to investigate the effect of deformation pattern on the bond-slip behavior under monotonic loading.

![Figure 17: Bar deformation pattern parameters [Pochanart & Harmon, (1989)]](image)

Pochanart and Harmon (1989) found the shape of the monotonic envelope was controlled primarily by the deformation pattern, which referred to as the lug spacing to lug height ratio, as well as the concrete compressive strength. Therefore, the developed model was quantified by these two parameters as shown in Figure 18 and the model could be described as:

\[ \tau = \tau_1 \left[ 1 - \frac{(s_1 - s)^3}{s_1^3} \right] s \leq s_1 \]

\[ \tau = \tau_3 - (\tau_1 - \tau_3) \times \left( \frac{s - s_2}{s_3 - s_2} \right) s_1 < s \leq s_3 \]

\[ \tau = \tau_3 \quad s > s_3 \]
The four controlling parameters were determined from fitting the pull-out test data:

- The peak bond stress, \( \tau_1 = 4.2 - 0.06 \frac{l_s}{l_h}; \tau \)
- The frictional bond strength, \( \tau_3 = 0.8 - 0.01 \frac{l_s}{l_h} \)
- The slip corresponding to the peak bond stress, \( s_1 = 0.003\sigma_b \)
- The slip corresponding to the onset of frictional bind resistance, \( s_3 = l_s \)

where \( l_s \) is the clear spacing between steel lugs, \( l_h \) is the lug height, and \( \sigma_b \) is the bearing pressure. The bearing pressure was calculated as the force applied to the test bar divided by the bearing area of the lugs. The relationships for the three controlling parameters \( \tau_1, \tau_3 \) and \( s_1 \) were only valid for 1 in. bar diameter and must be adjusted for concrete strength.

The ascending branch followed the cubic relationship and depended on the maximum bond stress as well as the slip corresponding to the maximum bond stress; the descending branch followed a straight line up to frictional bond stress. The horizontal frictional branch represented the remaining frictional stress after the bond strength was eliminated.

Figure 18: Local bond stress-slip model proposed by Pochanart & Harmon (1989)
It was observed that the slip corresponding to frictional bond stress \((s3)\) was roughly equal to the clear spacing between steel lugs. Also, the maximum bond strength and the frictional bond stress were typically insensitive to the deformation pattern, but the slip corresponding to the maximum bond strength was directly associated with the bearing pressure against the steel lugs. The slip corresponding to the peak bond stress is located between \(s1\) and \(s2\) in Eligehausen’s model. The peak bond stress \(\tau_1\) is smaller than the maximum bond stress of the bond-slip model proposed by Hawkins et al., 1982, but is greater than the peak bond stress of the bond-slip model proposed by Eligehausen et al., 1983. The test results suggested that the bond behavior could be improved by increasing the lug spacing about 1 to 1.25 times the bar diameter and by keeping the lug spacing-to-lug height ratio between 10 and 15. However, the effects of the scale were not included and the model developed was only applicable to a certain bar size and also required adjustment for concrete strength.

In the pullout test with a short embedment length, the bond-slip relationship was usually governed by the softening behavior of the concrete around the bar. As mentioned previously, the force applied to the bar was normally rather small compared to its capacity, and the reinforcing bar slip occurred when they were subjected to small strains. Shima et al. (1987), Mayer and Eligehausen (1998) suggested that bond condition of these bars may not be similar to those of fully anchored bars that experience high inelastic strains. New test procedures need to be designed in order to reach the yield load in the bars as well as to monitor the bond behavior until steel rupture.

### 2.4 Analytical Study of Bond Behavior of Deformed Bars

It is easily noticed that most research conducted on the bond-slip behavior presented so far is studied within the elastic range. The force applied to the bar is normally rather small compared to its capacity. However, the bond-slip relationship may change considerably when the deformed bar yields. As discussed previously, the bond failure is dominated by pullout failure for a reinforcing bar adequately anchored in a well-confined concrete block, where the main transfer mechanism changes from the rib bearing to the friction after the transverse cracks propagation is restrained by the sufficient confinement provided by transverse reinforcement. Under this condition, it is hypothesized that if the reinforcing bar
goes beyond yielding, the peak bond strength decreases and the bond stress drops more quickly due to the Poisson’s ratio effect, which results from the reduced contact area between lugs and concrete. This statement has been generally accepted by a number of researchers. In order to study the bond behavior after the deformed bar passes the yielding capacity, longer embedment length is normally required. Therefore, a uniformly distributed bond stress along the embedment length cannot be maintained. A typical method used to develop the local bond-slip model up to the inelastic strains was to conduct pullout tests with various long embedment lengths. Based on the experimental strain distribution along the embedment length corresponding to different loaded-end slip levels, the bar stress, local bond stress as well as local slip could be calculated. Therefore, a local bond-slip model which could satisfy the measured global response (represented by a relationship between the bar force and the loaded-end slip) could be derived. By using this technique, Engström et al. (1996) proposed a simplified bond-slip model for a deformed bar anchored in a well-confined concrete block. The proposed local bond-slip model is shown in Figure 19 ((I) deformed bar in the elastic range; (II) deformed bar in the plastic range). The proposed local bond-slip model for deformed bar in the elastic range (relationship I) is basically the same as in the MC90 model with the exception that the fourth branch representing the final frictional bond strength decreased linearly with the increase of local slip instead of being constant (frictional bond stress) as in the MC90 model. Engström et al. (1996) suggested that this final decreasing branch represented the gradual degradation of the interface in the final frictional phase. The parameters for normal strength concrete of relationship (I) are: $s_1 = 1.0$ mm, $s_2 = 3.0$ mm, $s_3=$ clear rib spacing, $s_4 = 3s_3$, $\tau_{\text{max}} = 0.45f_{\text{cm}}$, $\tau_f = 0.4 \tau_{\text{max}}$, and $\alpha = 0.4$. The relationship (II) in Figure 19 applied to the regions where the bar was strained beyond the yielding point. However, the value of the initial point $(s_y, \tau_y)$ of the second descending branch depended on the global response of the anchorage region and was calculated based on the force equilibrium and compatibility condition at each section. It was found that the local bond-slip relationship in the plastic range was different at each section along the anchorage length where the deformed bar passed the yielding point. Therefore, Engström et al. (1996) did not obtain a general local bond-slip model in the plastic range and it was very difficult to incorporate their model to any numerical analysis to simulate the global response.
Introducing a modification factor to the local bond stress is accepted as an effective way to account for the bond strength reduction in inelastic strains. Under this assumption, an analytical model based on finite element analysis was derived by Lowes through introducing a modification factor to account for the bond strength reduction due to the inelastic strain. In 2004, Lowes et al. developed a set of equations to determine the local bond stress-slip relationship. Concrete confining pressure, concrete damage state, steel strain, slip history, bar size, and concrete covered were all taken into account to the bond-slip model by applying different modification factors. Lowes et al. concluded that in correspondence to the peak bond strength, both the bond strength and slip decreased with an increase in bar strain. A modification factor that takes into account for the steel strains was proposed by Lowes et al. as:

\[
\Gamma_2 = \begin{cases} 
1.0 + 1.4 \left(1 - e^{\frac{0.4(1+\varepsilon_y)}{\varepsilon}}\right) & \varepsilon < -\varepsilon_y \\
1 & \varepsilon_y \geq \varepsilon \leq -\varepsilon_y \\
0.1 + 0.9 \left(e^{\frac{0.4(1-\varepsilon_y)}{\varepsilon}}\right) & \varepsilon > \varepsilon_y 
\end{cases}
\]  \tag{2.4}

The modification factor proposed by Lowes et al (2004) was examined by Wang (2008) through applying it to the Eligehausen’s local bond-slip model as a multiplier for the local bond stress and incorporating the revised bond-slip model to an analytical model. The analytical model incorporated three basic elements: a local bond stress-slip relationship proposed by Eligehausen et al. with Lowes et al.’s modification factor, a stress-strain relationship.
relationship for the steel (i.e., bilinear model), and force equilibrium as well as compatibility conditions between the reinforcing bar and surrounding concrete.

The simulated bar force vs. loaded-end slip relationship was compared to the experimental data derived from tests conducted by Viwathanatepa et al (1979). Wang concluded that the modification factor proposed by Lowes et al. could not properly represent the post-yield bond-slip behavior. Specifically, the modification factor excessively reduced the bond strength in the post-yield range. Therefore, a new modification factor was put forward by Wang (2008). Based on the experimental strain distribution along the embedment length corresponding to different loaded-end slip levels (Viwathanatepa et al., 1979), the bond stress, local slip as well as steel strain could be calculated at every middle point between the two adjacent strain gauges. Therefore, a local bond-slip model that could satisfy the measured global response (represented by a relationship between the bar force and the loaded-end slip relationship) and the strain distribution corresponding to different loaded-end slip levels could be derived. The modification factor was derived from curve fitting the derived local bond-slip model, while using Lowes’ modification factor as a reference. This modification factor was also deemed as a function of inelastic strain with same format as Lowes et al.’s modification factor, which could be expressed as:

\[ \varepsilon = 1 - 0.5 \left( 1 - e^{-0.5 \left( \frac{\varepsilon - \varepsilon_y}{\varepsilon_y} \right)} \right) \]  

(2.5)

The validity of this modification factor was examined by incorporating the improved local bond-slip model (Eligehausen et al.’s model with the modification factor proposed by Wang, 2008) to the same analytical model described previously to obtain the bar force vs. loaded-end slip relationship. The simulated response was compared to six groups of test data available in the literature, and the comparisons between the simulated response and two sets of test data are shown in Figure 20 and Figure 21. Based on these two figures, Wang proved that the proposed modification factor could improve the load-end slip relationship of deformed bar anchored in a concrete block, and hence improve the local bond-slip model in the post-yield range.
An alternative approach that maybe suitable for fiber-based analysis is to model the bar stress vs. slip hysteretic response directly as proposed by Zhao and Sritharan (2007), thereby...
capturing the local and global responses accurately. Zhao & Sritharan (2007) conducted a study to model the strain penetration effect of a longitudinal reinforcing bar anchored within a column-footing connection, where the longitudinal bar slip occurs only along a portion of the entire embedment length. Zhao and Sritharan noted that in flexural concrete members, strain penetration occurs along longitudinal reinforcing bars that are fully anchored into connecting concrete members, causing bar slips along a partial anchoring length and thus end rotations to the flexural members at the connection intersections. Ignoring the bar slips that result from strain penetration in linear and nonlinear analysis of reinforced concrete structures will underestimate the deflections and member elongation, as well as overestimate the member stiffness, hysteretic energy dissipation capacities, steel strains and section curvature. Strain penetration represents the gradual transferring of longitudinal bar forces to surrounding concrete in the connecting member. The slip exhibited by the connection interface of the loaded end of the anchored bar results from the accumulative strain difference between the bar and concrete within the connecting member which is characteristically different from slip caused by poor anchorage condition in the connecting member. The anchored bar in this case usually experiences much higher steel stress and steel strain level in comparison to the anchored bar in poor anchorage condition. It is critical to include the strain penetration effect in member modeling or analysis in reinforced concrete structures. A hysteretic model representing the reinforcing bar stress vs. slip response was proposed by Zhao & Sritharan (2007) based on limited measured bar stress and loaded-end slip from tests conducted on reinforcing bar anchored in concrete with sufficient embedment length available in the literature. The proposed monotonic bar stress vs. loaded end slip relationship consists of two parts: (1) a straight line for the elastic region and (2) a curvilinear portion for the post-yield region as shown in Figure 22.
Based on the discussion above, it is found that the bond behavior is a complicated problem, depending on various parameters such as bar size and geometry, concrete compressive strength, concrete confining capacity and anchorage condition. The existing experimental tests could not represent the bond behavior of deformed bars anchored in beam-column joints or column-footing connections accurately. Specifically, the test bars in single pull-out test with a short embedment length developed strains below the yield strain and the test bar slips occurred when they were subjected to strains well below the yield strain. This was possibly because the yield capacity of the deformed bar was far in excess of the total bond resistance that can be obtained within the short embedment length. Therefore, a longer embedment length was normally required to study the bond behavior in the plastic range. Up to now, most research conducted on the bond behavior in the plastic range was typically based on the finite element analysis and little experimental test has been conducted, especially for the column-footing connection where the longitudinal bar could develop strains that is significantly higher than the yield strain at the connection interface and the longitudinal bar could be pulled out with a significant ultimate slip at connection interface.

In conclusion, an experimental investigation need to be designed, which could simulate the bond slip behavior of a reinforcing bars anchored in a well-confined concrete block with
adequate embedment length as would be the case for the longitudinal bar anchored within a column-footing connection in which the strain penetration occurs only over a portion of the entire embedment length. The anchored longitudinal reinforcing bars are loaded at one end and the other end is set to be free. A detailed description of the test program is given in the next chapter.
CHAPTER 3. EXPERIMENTAL STUDY

This chapter provides details about the pullout test of the deformed bars completed for this thesis. The pullout test was used to simulate a reinforcing bar anchored in a well-confined concrete block with sufficient embedment length as would be the case for a longitudinal reinforcing bar embedded in a column-footing connection, column reinforcing bars extended into bridge joints, or reinforcement from girders anchored into cap beams. The measured response, which was represented by a relationship between the bar stress and the loaded-end slip, was compared with the predicted response to examine the accuracy of the hysteretic reinforcing bar stress vs. the loaded-end slip model developed by Zhao and Sritharan (2007). All specimens were cast and tested at the structural laboratory at Iowa State University. Two different sizes of deformed bars (i.e., #6 and #8) were tested under two different kinds of loading (i.e., monotonic and cyclic loading) and each individual test was identified with a unique name. The naming method summarized in Figure 23 is described as follows:

- the first term designates the type of reinforcement;
- the size of the deformed bar tested is denoted by the second term;
- the third term indicates the loading type; and
- the final term is used to distinguish between different tests with the same parameters.

<table>
<thead>
<tr>
<th>Type of Reinforcement</th>
<th>Size of Reinforcement</th>
<th>Loading Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>D = Deformed bar</td>
<td>6 = 0.75 in. dia. #6 deformed bar</td>
<td>M = Monotonic</td>
</tr>
<tr>
<td></td>
<td>8 = 1 in. dia. #8 deformed bar</td>
<td>C = Cyclic</td>
</tr>
</tbody>
</table>

For example, test specimen D.6.M.1 represents the first pullout test conducted on the #6 deformed bar under monotonic loading. A detailed description of the test program is given in the following sections.

3.1 Test Specimens

The test specimens in this series of tests were designed to capture the strain penetration effect. The specimens were detailed with adequate embedment length so that the test bar
could be fully developed. The test bars were fully anchored into a well-confined concrete block measuring 11 ft. by 16.5 in. by 48 in. and the concrete block was reinforced the same way as the bottom portion of an inverted-T cap beam (shaded area) as shown in Figure 24. Two types of reinforcement were provided: transverse and longitudinal. Thirteen No. 3 longitudinal bars were placed at the top and bottom portion of the specimen respectively, and seven No. 4 longitudinal bars were placed at the right and left middle portion of the specimen respectively. The clear concrete cover was approximately 1 inch. Adequate transverse reinforcement was provided to ensure the test bar would fail by fracture of the reinforcing bars. The specimen was designed to ensure that large inelastic strains could be obtained as the bar was pulled out.

Figure 24: Typical inverted-T cap beam reinforcement details (Snyder et al. 2011)

A single concrete block was designed with five test bars. The details of the reinforcing cages of the concrete block are shown in Figure 25. All of the test bars went all the way through the concrete block. A 12 in. long PVC tube was placed around the #6 test bars concentrically at the bottom end to debond along this region, which gave an embedment length of 36 in. (Figure 26). The embedment length for the #8 test bar was 48 in, which was the entire depth of the test specimen. The anchorage length for both types of test bars accounted to 48 bar diameters. The test bars were spaced in a zigzag manner: spaced at 18 in. in the longitudinal direction and 10.5 in. in the orthogonal direction. A photograph showing the reinforcing cage prior to casting is presented in Figure 27.
(a) Full view

(b) Side view
Figure 25: Reinforcing cage details of pullout test
Figure 26: Photograph showing #6 test bars with 12-in. de-bonded length at the end

Figure 27: Photograph showing the reinforcing cage prior to casting
3.2 Material Properties

3.2.1 Concrete

Ready mix concrete made of normal weight aggregate was used in this test. Standard compression tests were performed on samples of the test specimen’s concrete following each pullout test. Table 1 summarizes the concrete compressive strength, which was obtained using standard compression test procedures. The concrete compressive strength was defined as the uniaxial compressive strength of cylinders with 6 in. in diameter and 12 in. in height. The mean concrete compressive strength reported in Table 1 (i.e., 5.5 ksi) was later used in the analysis of test data.

Table 1: Compressive strength of concrete used in the pullout test

<table>
<thead>
<tr>
<th>Sample #</th>
<th>3 Days (psi)</th>
<th>7 Days (psi)</th>
<th>14 Days (psi)</th>
<th>28 Days (psi)</th>
<th>Day of Testing (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3423</td>
<td>4397</td>
<td>4933</td>
<td>5086</td>
<td>5562</td>
</tr>
<tr>
<td>2</td>
<td>3464</td>
<td>4412</td>
<td>4458</td>
<td>5218</td>
<td>5610</td>
</tr>
<tr>
<td>3</td>
<td>3694</td>
<td>4283</td>
<td>4933</td>
<td>5253</td>
<td>5473</td>
</tr>
<tr>
<td>MEAN</td>
<td>3527</td>
<td>4364</td>
<td>4775</td>
<td>5186</td>
<td>5548</td>
</tr>
</tbody>
</table>

3.2.2 Deformed bars

Grade 60 A706 deformed bars were used for both test bars and also as reinforcement of the reinforcing cages. The clear lug spacing for the #6 and #8 test bars is 0.5 in. and 0.75 in., respectively. A typical tensile monotonic stress-strain history of a Grad 60 A706 deformed bar is presented in Figure 28 and some important characteristics of this relationship are summarized as below:

1. Initial response is linear-elastic for stress demand that is less than the initial yield strength.

2. For strain demand exceeding that corresponding to the initial yield strength, there is a slight drop in strength below the initial yield strength. Strength is maintained at this lower yield strength for moderate increase in strain demand. This range of response is referred to as the yield plateau and the material yield strength is typically defined to be the average strength for loading within this strain range.
3. Increasing strain demand results in increased strength. This strain-hardening regime is maintained to a peak strength that typically exceeds the yield strength by 30 to 60 percent. The ratio of peak strength to nominal strength is a function of the steel specification, grade and batch composition.

4. At severe tensile strain demand, reinforcement begins to neck and strength is reduced.

5. At maximum strain demand, the steel reinforcement fractures and load capacity is lost.

**Figure 28**: Tensile monotonic stress-strain history for typical reinforcing steel bar  
(Data for A706 Grade 60 Reinforcement [Naito, 1999])

Tension tests were conducted on 12 in. long sample test bars to get the mechanical characteristics of the reinforcing bars. The sample test bars were loaded by a MTS uniaxial machine and the test bars were subjected to monotonically increasing tension load. Figure 29 shows the experimental stress-strain relationship of a #6 (Figure 29 a) and a #8 (Figure 29 b) test bar, respectively. The extensometer was removed at a corresponding strain of 0.025 in/in during the tension tests. After removing the extensometer, the load was continued to be applied, thereby increasing the bar stress until fracture.
(a) Experimental stress-strain relationship of #6 reinforcing bar

(b) Experimental stress-strain relationship of #8 reinforcing bar

Figure 29: Experimental stress-strain relationship of reinforcing bar subjected to tension tests
Table 2 represents the measured mechanical characteristics for the #6 and #8 test bars, which will be used in the analytical study in Chapter 5. It could be noted that both the #6 and #8 reinforcing bars have yield strength well above the specified value of 60 ksi for Grade 60 A706 deformed bars.

Table 2: Details of reinforcing bar mechanical characteristics measured in tension test

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Yield strength (ksi)</th>
<th>Yield strain (%)</th>
<th>Ultimate strength (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#6 (0.75 in. diameter)</td>
<td>76</td>
<td>0.233%</td>
<td>97</td>
</tr>
<tr>
<td>#8 (1 in. diameter)</td>
<td>67</td>
<td>0.209%</td>
<td>96</td>
</tr>
</tbody>
</table>

3.3 Test Matrix

Out of the five test bars, three of the bars were #6 with two subjected to monotonic tests and the remaining one subjected to cyclic test. The remaining two bars were #8 with one subjected to a monotonic test and the other subjected to a cyclic test. Table 3 provides a description of bar size, concrete compressive strength, embedment length and loading type of the pullout tests.

Table 3: Test matrix

<table>
<thead>
<tr>
<th>Test Bar</th>
<th>Concrete Compressive Strength: $\Gamma_c$ (ksi)</th>
<th>Embedment Length (in.)</th>
<th># of Specimen</th>
<th>Loading Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>#6 (0.75 in.)</td>
<td>5.5</td>
<td>36 (straight)</td>
<td>2</td>
<td>Monotonic</td>
</tr>
<tr>
<td>5.5</td>
<td>36 (straight)</td>
<td>1</td>
<td>Cyclic</td>
<td></td>
</tr>
<tr>
<td>#8 (1 in.)</td>
<td>5.5</td>
<td>48 (straight)</td>
<td>1</td>
<td>Monotonic</td>
</tr>
<tr>
<td>5.5</td>
<td>48 (straight)</td>
<td>1</td>
<td>Cyclic</td>
<td></td>
</tr>
</tbody>
</table>

3.4 Construction and Curing

The specimen was cast in an upright position in wooden forms. The concrete was casted in a vertical direction parallel to the test bars and the pullout force was applied in the opposite direction. The concrete was thoroughly vibrated through a plunger vibrator and finished with trowel to create a flat surface suitable for testing. The specimen was covered with plastic wrap to ensure a moist curing condition. The forms were removed approximately
seven days after casting. Figure 30 shows the specimen before casting and Figure 31 shows the specimen during casting.

Figure 30: Test specimen before casting

Figure 31: Test specimen in the middle of casting
3.5 Test Setup

Figure 32 presents the general test layout. The concrete block was post-tensioned to the floor through high-strength threaded rods at two ends that were served as reactions to the forces applied to the specimen. This test setup was designed to minimize the influence of the reactions that might have had on the response of the test bar when it was loaded in tension. The concrete block was supported by two I-shaped steel beams at each end and the test bar was connected to a threaded rod through a coupler. A movable frame was adopted to apply the load on each test bar. Load was applied to one end of the test bar through a capacity hydraulic ram and the other end was set to be free.

Figure 32: Pullout Test Setup
3.6 Test Instrumentation

Based on the discussion in chapter 2 (section 2.3), limited test data was available to provide direct local slip measurement along the reinforcing bar fully embedded in reinforced concrete. The local slip along the embedment length is usually obtained by taking the summation of the free end slip, \( s_0 \) and the integration of strain distribution along the embedment length from the free end, \( x_0 \) to point of interesting point, \( x \). The local slip at any point is then formulated as:

\[
s = s_0 + \int_{x_0}^{x} \varepsilon (x) dx
\]  

(3.1)

Therefore, the obtained slip depends greatly on the measured strain distribution, which may be affected by secondary cracks developed due to local failures occurred near the strain gauges. In order to obtain the direct local slip data along the embedment length, a new measurement was developed. A 3-D motion capture system designed and manufactured by Northern Digital Inc. (NDI) was employed in the pullout tests to receive real-time data (local slips, loaded-end strains and loaded-end slips) through its high-speed markers (i.e., LEDs). This Optotrak Certus Motion Capture System provided data with a higher level of accuracy and reliability compared to traditional data acquisition system. Three (for the #6 test bar) or four (for the #8 test bar) studs were attached to pre-selected target points along the embedment length (Figure 33) by welding them to the longitudinal test bars using Nelweld Model 6000 with NS 40N standard gun. The stud size was 3/8 in. diameter with a length of 3 inches. The current was set to 300 amps and the time was set to be 0.11 seconds, to ensure the minimum welding effect on the property of the test bar. The local slips could therefore be obtained directly from the readings of LEDs that were directly attached to the studs through the Optotrak Certus Motion Capture System. In order to have the stud move freely as the test bar was pulled out from the concrete block, a LED rod (i.e., an aluminum box) (Figure 34) was attached to the test bar around each stud. Proper amount of sealer was utilized to fill the gap between the LED rod and test bar, so as to prevent concrete flowing into the stud location during casting.
Figure 33: Photograph showing studs welded to the test bar along the embedded length
Figure 34: Photograph showing the LED rod

The location of the studs welded along each test bar is presented in Table 4.

Table 4: Stud locations along embedment length of each test bar

<table>
<thead>
<tr>
<th>Test Specimen</th>
<th>Stud Location (Distance from concrete block bottom in inches.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DM61</td>
<td>20 15/16, 32 15/16, 43 7/16</td>
</tr>
<tr>
<td>DM62</td>
<td>24 7/8, 34 5/8, 43 7/16</td>
</tr>
<tr>
<td>DC61</td>
<td>25 1/4, 34 1/4, 43 3/4</td>
</tr>
<tr>
<td>DM81</td>
<td>17 7/8, 30 7/16, 39 1/16, 44 11/16</td>
</tr>
<tr>
<td>DC81</td>
<td>18 1/8, 29 7/16, 38 7/8, 44 7/8</td>
</tr>
</tbody>
</table>
To validate the welding effect on the material property of test bars, tension tests were performed on both deformed bars with and without welding studs. Tension tests were again conducted on 12 in. long sample test bars. The tension test setup is shown in Figure 35. Figure 36 presents the measured stress-strain relationship of the #6 (Figure 36 a) and #8 (Figure 36 b) sample test bar with and without studs, respectively. Table 5 represents the yield strength and ultimate strength comparison between sample test bars with and without studs.

(a) With studs
Figure 35: Tension test setup of (a) sample deformed bar with studs (b) sample deformed bar without studs

(a) #6 sample deformed bar

(b) Without studs
Figure 36: Examining the influence of welded studs on stress-strain response of reinforcing bars

Table 5: Yield and ultimate strength of #6 and #8 deformed bar with and without studs

<table>
<thead>
<tr>
<th>Bar Type</th>
<th>Yield Strength (ksi)</th>
<th>Difference (%)</th>
<th>Ultimate Strength (psi)</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#6</td>
<td>76.8</td>
<td></td>
<td>97.8</td>
<td></td>
</tr>
<tr>
<td>#6 with stud</td>
<td>76.7</td>
<td>0.056</td>
<td>97.8</td>
<td>0.0023</td>
</tr>
<tr>
<td>#8</td>
<td>67.2</td>
<td></td>
<td>98.2</td>
<td></td>
</tr>
<tr>
<td>#8 with stud</td>
<td>67.7</td>
<td>0.816</td>
<td>96.9</td>
<td>1.35</td>
</tr>
</tbody>
</table>

Based on Figure 36, it could be observed that the stress-strain relationships for the reinforcing bars with and without studs correspond to each other very well for both #6 and #8 reinforcing bars. According to Table 5, the differences of the yield strength and ultimate strength between reinforcing bars with and without studs were also negligible, concluding that the welding effect on the material property of the reinforcing bars used in the pullout test conducted for this thesis can be ignored.
Four strain gauges (top strain gauge, top middle strain gauge, bottom middle strain gauge and bottom strain gauge) were placed along each test bar at pre-selected position (Table 6). Figure 37 shows the strain gauge placement on each test bar.

**Table 6: Strain gauge locations along embedment length of each test specimen**

<table>
<thead>
<tr>
<th>Test Specimen</th>
<th>SG Location (Distance from concrete block top surface in inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DM61</td>
<td>0, 6, 12, 24</td>
</tr>
<tr>
<td>DM62</td>
<td>0, 9, 18, 27</td>
</tr>
<tr>
<td>DC61</td>
<td>0, 9, 18, 27</td>
</tr>
<tr>
<td>DM81</td>
<td>0, 12, 24, 36</td>
</tr>
<tr>
<td>DC81</td>
<td>0, 12, 24, 36</td>
</tr>
</tbody>
</table>

**Figure 37: Photograph showing the strain gauge instrumentation on each test bar**

The primary quantities measured during the pullout test were: pullout forces applied to the test bars, the slips at both loaded-end and free end at each load level, the displacement at
each stud location along the embedment length, as well as the strains at the pre-selected locations. The strain distribution could be obtained from the readings of internal strain gauges. The forces and displacements were measured through external devices. The forces applied to the test bars were measured through a carefully calibrated load cell incorporated into a hydraulic ram. Two DCDTs (top and bottom) were adhered to each test bar with respect to the concrete block at both the loaded-end and free end to measure the slip relative to surrounding concrete. LED, which was attached to the stud that was welded to the test bars along the embedment length, was used to measure the local slip at each stud location. Besides measuring the local slip at each stud location along the embedded test bar, two additional LEDs (one was located at the top concrete block and the other was located at the bottom of concrete block) were directly attached to the surface of concrete block to measure its movement during the pullout test. As the test bar was pulled out, the concrete around the test bar was expected to crack and became loose at the loaded-end and the top strain gauge’s readings therefore could be affected. In order to get the strains as well as the slip at the concrete block and test bar interface, several LEDs were placed on the test bar beyond the anchorage region (Figure 38). One of these LEDs was located right on the top DCDT to confirm the displacement measured by the DCDT located at the loaded-end. Before attaching the LED to the test bar, the test bar beyond the anchorage region was cleaned with a wire brush and acetone to remove any rust on the rebar surface.
Figure 38: Photograph showing LEDs attached to the test bar above the interface

Based on test results from Test1 and 2 (as to be discussed in detail in next chapter), it was interesting to notice the scatter and discrepancy of the loaded-end strain calculated from LEDs mounted to the test bar beyond the anchorage region. One possible reason suspected to be due to the glue type and how LEDs were mounted to the test bar. In order to investigate the effect of glue type and the attachment method of LEDs to the test bar on the LED’s readings, an additional test was conducted.

This test was conducted on #6 deformed bar, which was subjected to increasing tension force applied by the MTS uniaxial machine until fracture. The sample of the #6 deformed bar was cleaned with a wire brush and acetone. The extensometer was attached to the rebar at a position between LED6 and LED5 and was removed at a strain level of 0.06 in/in during the test. The order of LEDs from top to bottom was: 11, 12, 9, 10, 7, 8, 6, 5, 3, 4, 1 and 2. Table 7 shows the glue type and how each LED was attached to the rebar. The NDI system
was set at a frequency of 30 Hz, while the MTS uniaxial machine was set at a frequency of 10 Hz. Figure 39 shows this reinforcing bar test setup.

Table 7: Glue type and attachment method of each LED

<table>
<thead>
<tr>
<th>LED #</th>
<th>Glue Type</th>
<th>How to attach to rebar?</th>
</tr>
</thead>
<tbody>
<tr>
<td>11</td>
<td>New³</td>
<td>Through nuts²</td>
</tr>
<tr>
<td>12</td>
<td>New³</td>
<td>Through nuts²</td>
</tr>
<tr>
<td>9</td>
<td>Black⁴</td>
<td>Through nuts²</td>
</tr>
<tr>
<td>10</td>
<td>Black⁴</td>
<td>Through nuts²</td>
</tr>
<tr>
<td>7</td>
<td>New³</td>
<td>Direct¹</td>
</tr>
<tr>
<td>8</td>
<td>New³</td>
<td>Direct¹</td>
</tr>
<tr>
<td>6</td>
<td>Regular</td>
<td>Direct¹</td>
</tr>
<tr>
<td>5</td>
<td>Regular</td>
<td>Direct¹</td>
</tr>
<tr>
<td>3</td>
<td>New³</td>
<td>Direct¹</td>
</tr>
<tr>
<td>4</td>
<td>New³</td>
<td>Direct¹</td>
</tr>
<tr>
<td>1</td>
<td>Black⁴</td>
<td>Through nuts²</td>
</tr>
<tr>
<td>2</td>
<td>Black⁴</td>
<td>Through nuts²</td>
</tr>
</tbody>
</table>

Notes:
1. “Direct” meaning the LED is directly attached to the rebar through glue;
2. “Through nuts” meaning the LED was attached to the head of nut through hard glue and the nut was then attached to the rebar through different glue type;
3. New glue is Dow Corning 3145 RTV “Silicone based adhesive”;
4. Black glue is Loctite 410 and Loctite 7452 used together.
The strain calculated based on each of the two LEDs’ displacements is plotted in Figure 40 and is compared to the expected strain experienced by the rebar during the test. The expected strain was measured by the extensometer located between LED6 and LED5. Since LED11 dropped off from the sample test bar and stopped working before the test finished, the strain calculated from LED11 and LED12 is not included in Figure 40. Based on this figure, it could be noticed that the strain calculated from LED6 and LED5, LED7 and LED8, as well as LED3 and LED4 produced a comparable strain history to that of expected strain measured by the extensometer, especially the strain calculated from LED6 and LED5 as well as LED3 and LED4. All of these LEDs were directly attached to the test rebar. LED6 and LED5 were attached to the rebar directly through hard glue and the extensometer was located at the position between LED6 and LED5. LED3 and LED4 were attached to the rebar directly through new glue and were located below the extensometer. LED7 and LED8 were attached to the rebar directly through new glue and were located above the extensometer.
One important thing need to be noticed is that the strain calculated from LED3 and LED4 had a delay for the test bar going to the yielding point.

It was very interesting to notice in Figure 40 that the strain calculated from LED9 and LED10, (which were attached to the rebar through nuts using black glue and were located above the extensometer) went to negative as the rebar experienced the initial yielding. The strain calculated from LED1 and LED 2, (which were attached to the rebar through nuts using black glue and were located below the extensometer) had a delay for the rebar going to the yielding. These two situations are the same as what were observed in Test 1 and Test 2, in which the LEDs were attached to the test bar through nuts.

Based on the results observed in the LED verification test, it was concluded that the strain calculated from the LEDs directly attached to the rebar agrees with the expected strain better compared to the strain calculated from LEDs that were attached to the rebar through nuts, regardless of the glue type used. The glue type did not have a significant effect on the strain calculation. This LED verification test concluded that the tests conducted following Test 1 and Test 2 should use LEDs directly attached to the test bar.

![Figure 40: Strain history comparison of LED verification test](image-url)
3.7 Loading History

As previously noted, the specimens were subjected to two types of loading: monotonic and cyclic loading. Load was only applied to one end of the test bar through a hand-operated hydraulic pump, which could be controlled by either load or displacement and the other end was set to be free. In the monotonic experiments, the test bars were pulled out under increasing monotonic tension forces under force control before yielding and then switched to loaded-end displacement control after yielding until the bars fractured. The load was applied in 5 steps until the test bar experienced yielding and in a number of deformation steps after the test bar yielded with the deformation load step adjusted during each test. In the cyclic experiments, the test bar was loaded under half cycle to each specified force level with 4 load steps in elastic range and was loaded under three half cycles to each specified loaded-end displacement measured by the DCDT located at the loaded-end with 6 deformation steps after the bar passed the yielding point until the test bar fractured as listed in the Table 8. The pre-selected target displacements were listed as sy, 3sy, 6sy, 9sy, 12sy, 16sy, 20sy, 25sy and 30sy, where sy was the loaded-end displacement measured by the top DCDT corresponding to yield strength of test bars during the test. Both the NDI system and the data acquisition system were set at a frequency of 10Hz. The NDI system was set in the x-y plane with the origin located at the test bar and concrete block interface. The y-positive direction was above the interface and the y-negative direction was below the interface. All the pullout tests conformed to this rule and all bars were tested until failure that was caused by fracture of the test bars.
### Table 8: Loading path of pullout test under cyclic loading

<table>
<thead>
<tr>
<th>Reinforcing bar</th>
<th>#6</th>
<th>#8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic range</td>
<td>Target Force (kips)</td>
<td>Target Force (kips)</td>
</tr>
<tr>
<td></td>
<td>8.3</td>
<td>13</td>
</tr>
<tr>
<td></td>
<td>16.7</td>
<td>26</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>39</td>
</tr>
<tr>
<td></td>
<td>33</td>
<td>52</td>
</tr>
<tr>
<td>Inelastic range</td>
<td>Target Displacement (in)</td>
<td>Target Displacement (in)</td>
</tr>
<tr>
<td></td>
<td>0.02</td>
<td>0.037</td>
</tr>
<tr>
<td></td>
<td>0.06</td>
<td>0.11</td>
</tr>
<tr>
<td></td>
<td>0.12</td>
<td>0.22</td>
</tr>
<tr>
<td></td>
<td>0.18</td>
<td>0.33</td>
</tr>
<tr>
<td></td>
<td>0.24</td>
<td>0.44</td>
</tr>
<tr>
<td></td>
<td>0.32</td>
<td>0.59</td>
</tr>
<tr>
<td></td>
<td>0.4</td>
<td>0.73</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td>0.6</td>
<td>1.1</td>
</tr>
</tbody>
</table>
CHAPTER 4. PULLOUT TEST RESULTS

The purpose of this chapter is to present, analyze, and discuss the pullout test results. The data presented in this chapter are from the laboratory tests and will be analyzed in three different ways: (1) bar stress vs. loaded-end slip relations; (2) strain distribution along the embedment length; and (3) local slip distribution along the embedment length.

4.1 Overview

The strain penetration test results will be presented with two classifications: monotonic loading and cyclic loading. The main results can be classified as follows:

- Bar stress vs. loaded-end slip relations - this curve shows the overall general behavior of test bar embedded in a confined concrete block with a sufficient embedment length;
- Strain distribution along the embedment length with loaded-end strains well above the yield strain of the test bar; and
- Local slip distribution along the embedment length with loaded-end strains well above the yield strain of the test bar.

All of the results can be obtained readily from the test data. In the following data analysis sections, LED represents the LED readings corresponding to each load level; LED_0 is the LED initial reading corresponding to zero load, which gives the initial location of the LED with respect to the test bar and concrete block interface. The loaded-end strain, as well as the loaded-end slip, is the strain and slip measured at the concrete block and test bar interface, respectively.

4.2 Visual Observations

4.2.1 Monotonic loading

In the monotonic tests, considerable cracks occurred on the concrete surface at the loaded end, while no visible cracks were observed at the free end. For a #6 test bar, the first splitting tension crack on the top surface of the concrete block occurred at a stress level of 54 ksi. As the load increased, more splitting cracks on the concrete surface were either formed or extended from previous splitting cracks. After the #6 test bar went past the yielding stress,
the test was switched to target the displacement measured by the DCDT located at the loaded-end. The test bar began to be pulled out at a corresponding loaded-end displacement of 0.5 inches, and was pulled out further as the loaded-end displacement increased. There were no visible cracks formed on the side surface. The test bar finally fractured with a corresponding stress of 97 ksi.

A total of four pullout tests were conducted on the same #8 rebar. Three attempts were made before the test bar finally fractured during the fourth attempt. Three previous attempts failed right after the test bar went past the yielding point due to coupler slipping with respect to the test bar. The test bar was damaged due to the coupler’s slipping and the test had to be stopped since the load could not be continued to be applied on the test rebar. The fourth attempt was conducted on the same test bar as before after cutting the damaged portion of the bar, while also adopting a coupler filled with grout between the coupler and the test bar. This was done to ensure the force could continue to be applied until the bar rupture and also the coupler could transfer the force from the jack to the test bar satisfactorily. The entire test lasted about one hour. The first splitting tension cracks occurred at stress of 63.3 ksi. Three cracks occurred on the top surface with one crack extending to the side and stopping at the first stud location (Figure 41). After the test bar went past the yielding point, the test was switched to target loaded-end displacement measured by the DCDT. The cracks in the concrete surrounding the test bar at the loaded-end formed the shape of a cone. A slight pullout around the rebar initiated at a target displacement of 0.2 inches. As the loaded-end displacement increased, an obvious test bar pullout could be observed and several large cracks occurred on the side surface, resulting in an evident cone shape (Figure 42). The concrete cone completely fractured away from the rest of the block and the test bar finally fractured at a stress level of 97 ksi.

4.2.2 Cyclic loading

Compared to the monotonic loading, more extensive cracks were developed under cyclic loading. An obvious splitting of cover concrete was observed and significant piece of concrete block completely fractured away from the rest of the block before test bars fractured. The cracks developed under cyclic loading followed the same trend as monotonic loading.
Figure 41: First splitting crack extended to the side surface of #8 test bar subjected to monotonic loading

Figure 42: A coned shape formation of concrete surrounding the #8 test bar subjected to monotonic loading
For the #6 test bar, the splitting of cover concrete formed when the loaded-end displacement reached to 0.29 inches. The cracks on the side surface extended even further when the loaded-end displacement was increased to 0.44 inches (Figure 43). A more extensive bar pullout was observed in comparison to monotonic loading (Figure 44). The test bar fractured with some distance inside the concrete block at a stress level of 97 ksi (Figure 45).

Figure 43: Photograph showing splitting of cover concrete of the #6 test bar subjected to cyclic loading
Figure 44: Photograph showing pullout of the #6 bar subjected to cyclic loading

Figure 45: Photograph showing the #6 bar fractured at some distance inside the concrete block during cyclic loading
The bar stress vs. loaded-end slip relationship provides the most significant data to evaluate the overall performance of a reinforcing bar anchored in a well-confined concrete block. The bar stress could be readily obtained through the applied force divided by the nominal cross sectional area of the rebar. Two DCDTs were used to measure the displacement at both the free end and also the loaded end. Based on the test data, there was no obvious visual movement presented at the free end during the entire pullout test, which confirmed that the embedment length was sufficient for the test bar to be fully developed up to the ultimate strength. The slip measured at the concrete block to test bar interface was taken as the loaded-end slip, which could be derived by subtracting the elongation of test bar from either the displacement of each LED located above the interface or the displacement measured by the DCDT located at the loaded-end. The elongation of the test bar was calculated as the loaded-end strain multiplied by the distance from the interface to the location of each LED or the location of the DCDT at the loaded-end. Therefore, the loaded-end strain was calculated first before plotting the bar stress vs. loaded-end slip relationship.

4.3 Loaded-End Strain Calculation

4.3.1 Monotonic loading

Although a top strain gauge was placed at the concrete block and test bar interface to measure the loaded-end strain, it did not work properly up to the point in which the test bar fractured. As previously discussed, the top strain gauge readings were not reliable and should not be used as the loaded-end strain in the following data analysis. The loaded-end strains used in the data analysis were calculated based on the displacement of two LEDs placed above the interface (such as LED12 and LED1) and could be expressed as:

\[
\text{Strain}_{\text{loaded-end}} = \frac{(LED_{12} - LED_1) - (LED_{12i} - LED_{1i})}{LED_{12i} - LED_{1i}}
\]  

(4.1)

The average loaded-end strain calculated from all the LEDs located above the interface was used as the loaded-end strain.

Figure 46 shows the loaded-end strain comparison of the #6 test bar under monotonic loading. The #6 monotonic test was conducted before the LED verification test was
conducted, so the LEDs placed beyond the anchorage region were mounted to the test bar through nuts that were attached to the test bar by the use of black glue.

![Figure 46: Loaded-end strain comparison of a #6 test bar subjected to monotonic loading](image)

Based on the Figure 46, it is interesting to notice that the strain calculated from LED9 and LED4 starts to decrease and then drops to negative values as the test bar experienced yielding. This situation was the same as what had been noticed in the LED investigation test. It is obvious that the loaded-end strain should never go to negative values at all load levels. The loaded-end strain calculated from LED1 and LED2 as well as LED2 and LED9 seemed to correspond well with each other in a similar trend. Therefore, it was concluded that the data obtained from LED4 was not reliable and should not be used in the data analysis. With respect to the strain calculated from LED1 and LED2 as well as LED2 and LED9, one obvious discrepancy occurred when the test bar reached the yield limit. The strain calculated from LED1 and LED2 seemed to have a delay in arriving at the yielding point, compared to the strain calculated from LED2 and LED9. One possible reason for this difference may be due to use of different glue types and the LED attachment methods. A LED verification test was therefore conducted under this assumption. Because the data from LED4 could not be
used in the data analysis, the strain calculation from LED9 and LED4 were not used. Figure 47 shows the average strain calculated from LED1 and LED2 as well as LED2 and LED9. As can be seen from Figure 47, there is a big increase in the loaded-end strain as the #6 test bar experienced the initial yielding. The ultimate strain corresponding to the bar fracture was around 0.062 in/in.

Figure 47: Loaded-end strain history of a #6 test bar subjected to monotonic loading

Figure 48 presents the loaded-end strain comparison conducted on the #8 test bar that was subjected to monotonic loading. Based on the test results from the LED investigation test, it was concluded that the strain calculated from the LEDs placed at the location of extensometer provided a more precise strain with respect to the extensometer readings compared to the other LEDs. Therefore, the strain calculated from LED8 and LED11 should give a more satisfactory loaded-end strain than the other LEDs, since the loaded-end strain was measured at the interface and LED8 and LED11 were placed closer to the interface than the other LEDs. The loaded-end strain calculated from LED9 and LED12, LED12 and LED8 as well as LED8 and LED11 were comparable with each other, especially those calculated from LED9 and LED12 as well as LED12 an LED8. Deleted the loaded-end strain calculated from LED10 and LED9 and averaged the remaining ones, providing Figure 49. According to Figure 49, there is no sudden increase of loaded-end strain for the #8 test bar as
has been observed for the #6 test bar. The likely reason for this observation is that three pullout test attempts were made before the #8 test bar fractured in the fourth pullout test and the strain hardening had already occurred in the three previous attempts. Figure 49 plots the loaded-end strain history of the #8 test bar in the fourth attempt only. The ultimate strain corresponding to the #8 test bar’s fracture was around 0.058 in/in.

![Graph of loaded-end strain history](image)

**Figure 48: Loaded-end strain comparisons of #8 test bar subjected to monotonic loading**

### 4.3.2 Cyclic loading

The loaded-end strain was calculated similarly for cyclic loading as it was for monotonic loading. Figure 50 presents the loaded-end strain calculated based on two LEDs placed above the concrete block and test bar interface for the #6 test bar. The loaded-end strain that was taken as the average of all possible strain values given in Figure 50 for the #6 test bar subjected to cyclic loading is shown in Figure 51. For the cyclic loading, a significant increase in loaded-end strain occurred after the test bar passed the yielding point. In addition, a substantial increase in loaded-end strain occurred as the test bar experienced the initial first half cycle at each pre-selected target loaded-end displacement.
The ultimate loaded-end strain for the #6 deformed bar subjected to cyclic loading was around 0.078 in/in, which is greater than that of the same test bar subjected to monotonic loading (0.062 in/in).

The loaded-end strain for the #8 test bar subjected to cyclic loading is shown in Figure 53. The ultimate loaded-end strain is around 0.08 in/in, which is also greater than the ultimate loaded-end strain experienced by the same test bar subjected to monotonic loading (0.057 in/in). However, the ultimate loaded-end strains for both the #6 (0.078 in/in) and #8 (0.08 in/in) test bars with the same ultimate strength subjected to cyclic loading was approximately the same. A significant increase in loaded-end strain occurred after the test bar passed the yielding point. The loaded-end strain history for the #8 test bar was developed following a similar trend as that of the #6 test bar.

Compared to the monotonic loading, the test bar under cyclic loading produced a larger ultimate loaded-end strain. The ultimate loaded-end strain was around the same value for both the #6 and #8 test bars subjected to either monotonic loading (0.06 in/in) or cyclic loading (0.08 in/in).
Figure 50: Loaded-end strain comparison of the #6 test bar subjected to cyclic loading

Figure 51: Loaded-end strain history of the #6 test bar subjected to cyclic loading
Figure 52: Loaded-end strain comparison of the #8 test bar subjected to cyclic loading

Figure 53: Loaded-end strain history of the #8 test bar subjected to cyclic loading
4.4 Loaded-End Slip Calculation

4.4.1 Monotonic loading

Once the loaded-end strain was obtained, the loaded-end slip calculated based on the displacement of each LED located above the interface could be obtained and was expressed as:

\[
\text{Slip}_{\text{Loaded-end}} = \text{LED} - \text{Strain}_{\text{Loaded-end}} \times d - d_{\text{concrete}}
\]

(4.2)

where \(d\) is the distance from the LED to the interface and \(d_{\text{concrete}}\) represents the concrete movement with respect to test bar and concrete block interface. Based on this loaded-end slip equation, the loaded-end slip calculated from each LED located above the interface of both the #6 and #8 test bars subjected to monotonic loading is plotted in Figure 54 and Figure 56, respectively. Based on these two figures, it is observed that the loaded-end slip calculated from each LED placed above the interface agrees to each other pretty well. Averaging the loaded-end slip calculated from all the LEDs of both the #6 and #8 test bars provided the loaded-end slip history as shown in Figure 55 and Figure 57, respectively. The ultimate loaded-end slip for the #6 test bar was around 0.43 inches, while the ultimate loaded-end slip for the #8 test bar was around 0.83 inches. Based on limited pullout test data available in the literature for deformed reinforcing bars with sufficient embedment length, the loaded-end slip corresponding to the yield bar stress was established by Zhao and Sritharan (2007), which is represented below:

\[
s_y = 2.54 \left( \frac{d_y (\text{mm})}{8437} \right) \frac{f_y (\text{MPa})}{\sqrt{f'_c (\text{MPa})}} (2\alpha + 1)^{1/\alpha} + 0.34
\]

(4.3)

where \(\alpha\) is the parameter used in the local bond-slip relation and was taken as 0.4 in this study in accordance with the CEB-FIP Model Code 90. From this equation, the yield slip for the #6 and #8 test bar is 0.02 inches and 0.024 inches, respectively. Zhao and Sritharan suggested that the ultimate loaded-end slip would be in the range of 30 to 40 times the yield loaded-end slip based on the test information. For an easy analysis, the ultimate loaded-end slip was taken as 35 times the yield loaded-end slip in the following data analysis. Therefore, the loaded-end slips at the bar ultimate strength for the #6 and #8 test bar was expected to be
0.7 inches and 0.84 inches, respectively. Table 9 and Table 10 presents the loaded-end slip corresponding to the yield and ultimate strength of the #6 as well as the #8 test bar, respectively. Compared to the test data, the equation proposed by Zhao and Sritharan gave smaller yield loaded-end slips for both the #6 and #8 test bars. The measured ultimate loaded-end slip for the #6 test bar was 0.43 inches, which was much smaller than 0.7 inches given by the equation proposed by Zhao and Sritharan. However, it was not the case for the #8 test bar. The ultimate loaded-end slip equation proposed by Zhao and Sritharan gave a comparable value as that obtained from the pullout test results for the #8 test bar. Zhao and Sritharan underestimated the loaded-end slip corresponding to the yield strength by 36.3%. With respect to the ultimate loaded-end slip, the model proposed by Zhao and Sritharan was consistent with the value measured for the #8 test bar, but overestimated it for the #6 test bar. The difference may come from the fact that there was no substantial concrete cracking observed for the #6 test bar at the loaded-end, which meant part of the bond strength between the #6 test bar and its surrounding concrete was still sustained before the #6 test bar fractured. However, separation of cover concrete was observed for the #8 test bar and a small concrete block was fractured away from the rest of the concrete block as the bar stress approached the ultimate strength. Therefore, the bond between the #8 test bar and surrounding concrete was completely lost at the loaded-end, leading to the reduction of the embedment length and slipping of the test bar. Zhao and Sritharan’s model did not include the state of the concrete cracking at the loaded-end and their model assumed the bond at the unconfined concrete region was lost depending on the observations of limited test data their model based on.

Table 9: Loaded-end slip at yield and ultimate strength comparison for the #6 test bar

<table>
<thead>
<tr>
<th>Loaded-end slip</th>
<th>Pullout test</th>
<th>Zhao and Sritharan’s model</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$s_y$</td>
<td>0.034</td>
<td>0.02</td>
<td>41.18</td>
</tr>
<tr>
<td>$s_u$</td>
<td>0.43</td>
<td>0.7</td>
<td>62.79</td>
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</table>
Table 10: Loaded-end slip at yield and ultimate strength comparison for the #8 test bar

<table>
<thead>
<tr>
<th>Loaded-end slip</th>
<th>Pullout test</th>
<th>Zhao and Sritharan’s model</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$s_y$</td>
<td>0.035</td>
<td>0.024</td>
<td>31.43</td>
</tr>
<tr>
<td>$s_u$</td>
<td>0.83</td>
<td>0.84</td>
<td>1.2</td>
</tr>
</tbody>
</table>

Figure 54: Loaded-end slip comparison of a #6 test bar subjected to monotonic loading
Figure 55: Loaded-end slip history of a #6 test bar subjected to monotonic loading

Figure 56: Loaded-end slip comparisons of the #8 test bar subjected to monotonic loading
4.4.2 Cyclic loading

The loaded-end slip history obtained from pullout test conducted on the #6 and #8 test bar subjected to cyclic loading is shown in Figure 59 and Figure 61, respectively. Same as the loaded-end strain, significant increase in the loaded-end slip also occurred at the first half cycle with respect to each target loaded-end displacement. The loaded-end slip at yield and ultimate strength of both the #6 and #8 test bar are presented in Table 11 and Table 12, correspondingly. The loaded-end slip at the yield strength for the #6 test bar under cyclic loading was comparable to that under monotonic loading, but it was not the case for the #8 test bar. The yield loaded-end slip for the #8 test bar under cyclic loading was much smaller than that obtained under monotonic loading. The ultimate loaded-end slip for both the #6 and #8 test bars under monotonic loading was significantly smaller than that under cyclic loading.

Figure 57: Loaded-end slip history of the #8 test bar subjected to monotonic loading

![Graph showing loaded-end slip history](image)
Table 11: Comparison of loaded-end slip at yield and ultimate strength between monotonic loading and cyclic loading of the #6 test bar

<table>
<thead>
<tr>
<th>Loaded-end slip</th>
<th>Monotonic loading</th>
<th>Cyclic loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>$s_y$</td>
<td>0.034</td>
<td>0.032</td>
</tr>
<tr>
<td>$s_u$</td>
<td>0.43</td>
<td>0.94</td>
</tr>
</tbody>
</table>

Table 12: Comparison of loaded-end slip at yield and ultimate strength between monotonic loading and cyclic loading of the #8 test bar

<table>
<thead>
<tr>
<th>Loaded-end slip</th>
<th>Monotonic loading</th>
<th>Cyclic loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>$s_y$</td>
<td>0.035</td>
<td>0.02</td>
</tr>
<tr>
<td>$s_u$</td>
<td>0.83</td>
<td>1.31</td>
</tr>
</tbody>
</table>

Figure 58: Loaded-end slip comparison of the #6 test bar subjected to cyclic loading
Figure 59: Loaded-end slip history of the #6 test bar subjected to cyclic loading

Figure 60: Loaded-end slip comparison of the #8 test bar subjected to cyclic loading
4.5 Bar Stress vs. Loaded-End Slip Diagrams

4.5.1 Monotonic loading

The bar stress vs. loaded-end slip under monotonic increasing tension force of the #6 and #8 test bars are shown in Figure 62 and Figure 63. It is noticed that significant degradation in stiffness took place right after the test bar reached the yielding limit. After the test bar passed the yielding point, no significant loss in strength or stiffness could be observed. Comparable yield and ultimate strength of the test bar were developed with respect to the specified values. For the #6 test bar, the bar stress decreased somewhat before it fractured. For the #8 test rebar, it was noted that the fourth pullout test produced slightly higher yield strength and less stiffness ascending branch in elastic range, compared to the previous three tests. This could be explained by the fact that strain hardening had occurred in the previous tests. The bar stress reached the ultimate strength before the bar fractured. From these particular test results, it is clear that 48d_b (d_b is the bar diameter) embedment length is sufficient for the test rebar to develop the ultimate strength.
Figure 62: Measured bar stress vs. loaded-end slip relationship of the #6 test bar

Figure 63: Measured bar stress vs. loaded-end slip relationship of the #8 test bar
In order to study the effect of bar size on the bar stress vs. loaded-end slip relationship, the monotonic bar stress vs. loaded-end slip relationship of both the #6 and #8 test bar are plotted in the same figure (Figure 64). Compared to the #6 test bar, the #8 test bar in the fourth pullout test produced a lesser stiffness ascending branch in the elastic range. The #8 test bar in the previous three pullout tests had a comparable stiffness with respect to the #6 test bar in the elastic range, which could be explained by the fact that the #8 test bar experienced strain hardening in the previous tests. The #8 test bar also developed a much larger loaded-end slip when the test bar experienced fracture. The ultimate slips for the #6 and #8 test bars were 0.42 inches and 0.82 inches, with corresponding loaded-end strains of 0.062 in/in and 0.058 in/in, respectively. In addition, the loaded-end slip corresponding to the maximum bar stress was approximately 0.2 inches for the #6 test bar and was 0.8 inches for the #8 test bar. The #8 bar stress was able to sustain at a stress of 97 ksi before it fractured, while the #6 bar stress decreased its resistance a little before it fractured.

Figure 64: Bar stress vs. loaded-end slip relationship comparison of #8 test bar
In order to validate the bar stress vs. loaded-end slip model proposed by Zhao and Sritharan, the bar stress vs. loaded-end slip relationship from both pullout tests, as well as the model, are plotted in the same figure. For the #6 test bar (Figure 65), the measured bar stress vs. loaded-end slip relationship corresponded to Zhao and Sritharan’s model pretty well in the elastic range. After the #6 test bar passed the yield point, the stiffness of the experimental response was a little greater than those observed in the model proposed by Zhao and Sritharan. In addition, the experimental test had an ultimate loaded-end slip that is much smaller compared to that proposed by Zhao and Sritharan. However, this is not the case for the #8 test bar (Figure 66). The bar stress vs. loaded-end slip relationship for the #8 test bar is plotted based on the fourth pullout test only. As discussed previously, since the strain hardening had occurred in the previous pullout tests, the fourth pullout test data showed greater yield strength and a relatively lesser stiffness ascending branch in the elastic range. Other than these two differences, the bar stress vs. loaded-end slip relationship obtained from the pullout test agreed pretty well with the proposed model for the #8 test bar. One difference between the monotonic test conducted on the #6 and #8 test bar came from the concrete cracking state at the loaded end (evident concrete cracking around the #8 test bar occurred at the loaded-end, while no substantial concrete cracking around the #6 test bar occurred at the loaded-end). By revisiting the test data used to develop the bar stress vs. loaded-end slip model proposed by Zhao and Sritharan, it was noticed that the embedment length for #6 deformed bars in those data set was in the range of 32 to 40 bar diameter, which was relatively smaller than that used in the pullout test (48 bar diameter). Also, evident cone formation was observed in those pullout tests conducted on the #6 deformed reinforcing bar. Therefore, it was concluded that the model proposed by Zhao and Sritharan could satisfactorily represent the bar stress vs. loaded-end slip relationship for pullout tests conducted on a deformed reinforcing bar anchored in a well-confined concrete block with adequate embedment length.
Figure 65: Bar stress vs. loaded-end slip relationship comparison between pullout test and Zhao and Sritharan’s model for a #6 test bar subjected to monotonic loading.

Figure 66: Bar stress vs. loaded-end slip relationship comparison between pullout test and Zhao and Sritharan’s model for the #8 test bar subjected to monotonic loading.
4.5.2 Cyclic loading

The bar stress vs. loaded-end slip relationship for the #6 and #8 test bars subjected to both monotonic and cyclic loading are presented in Figure 67 and Figure 68. Based on these two figures, it was observed that unloading and reloading at stress levels that are well above the rebar yielding stress produced no significant loss in strength or stiffness.

Figure 67: Bar stress vs. loaded-end slip relationship of the #6 test bar
4.6 Strain Distribution Diagrams

As noted in the test instrumentation section of chapter 3, there were four strain gauges mounted to the test bar along the embedment length. That section presented the strain gauge locations for each test rebar. Based on the test data, it was noted that the top strain gauge, (located right at the interface used to measure the loaded-end strain), did not generally work very well. Cracking occurred around the test bar at the loaded-end as the load increased, this resulted in the top strain gauge separating from the test bar and therefore stopped working at a very early stage. Hence, the top strain gauge readings were not reliable. Consequently, the average loaded-end strain calculated from the LEDs located above the interface was used as the loaded-end strain, as had been illustrated in the loaded-end strain calculation section. Figure 69 and Figure 70 present the measured bar stress vs. loaded-end strain relationship for the #6 and #8 test bar, respectively. In order to compare the loaded-end strain calculated based on the LEDs located above the interface to the actual strain, the bar stress vs. strain relationship derived from the material tests are also plotted in the same figures. At the initial
load level, the loaded-end strains of the #6 test bar experienced negative values, which could be a result of the LED attachment method used in Test 2 (Monotonic test conducted on the #6 test bar). As noted in the test instrumentation section of chapter 3, LEDs located above the interface were indirectly attached to the #6 test bar in Test 2, and thus these LED’s readings might not reflect the actual test bar movement under the applied load. Therefore, the loaded-end strain calculated based on these LED’s readings would have had inevitable scatters leading to errors in the loaded-end strain calculation. After the #6 test bar reached the yielding point, the relation derived from the pullout test and the material test appeared to correspond to each other very well, except that the loaded-end strain corresponding to yield stress of pullout test was a little smaller than that obtained from the material test. For #8 test bar, it was noted that the bar yield stress from pullout test was much greater than the actual yield bar stress. The bar stress vs. strain relationship obtained from #8 bar pullout test did not have an obvious strain hardening stage, which validates that the strain hardening occurred in previous tests.

Figure 69: Stress vs. loaded-end strain relationship of the #6 test bar
The strain distribution observed from strain gauge mounted to the #6 and #8 test bars along the embedment length subjected to monotonic loading are presented in Figure 71 and Figure 72. During the pullout of the test bar, the strain along the embedment length was recorded by each strain gauge at different loaded-end slip levels. As discussed earlier, the readings from the top strain gauge should not be used in the data analysis. Therefore, the loaded-end strain was plotted as the strain calculated from the LEDs located above the interface. The horizontal axis “position” indicated the embedment length from the free end of the test bar. Based on the measured strain distribution curve of both the #6 and #8 test bars, it could be noted that the strain increased as the loaded-end slip level increased. In addition, the strains with respect to different loaded-end levels did not differ from each other significantly in elastic range and the significant difference occurred after the test bar went past the yielding point.

**Figure 70: Stress vs. loaded-end strain relationship of the #8 test bar**
Figure 71: Measured strain distribution along the embedment length of the #6 test bar corresponding to 0.06, 0.08, 0.1 and 0.15 inches of loaded-end slip

Figure 72: Measured strain distribution along the embedment length of the #8 test bar corresponding to 0.1, 0.2, 0.3 and 0.4 inches of loaded-end slip
4.7 Local Slip Distribution Diagrams

The local slip distribution diagram describes the test bar movement respective to the concrete block. As discussed previously, limited test data was available to quantify the local slip distribution along the rebar directly. The local slip at the desired section \( x_i \) is typically obtained by integrating the strain distribution along the bar from the free end to the interesting section \( x \), plus the free end displacement, if any. In the pullout test conducted in this thesis, a new method (as described in chapter 3: test instrumentation section) was adopted to measure the local slip directly at a pre-selected location along the embedment length.

The local slip is defined as the relative movement of the test bar with respect to the surrounding concrete, which could be expressed as follows:

\[
S = LED - LED_i - d_{\text{concrete}}
\]

The measured local slip could be measured directly by the LED attached to the stud. The measured local slip at each stud location corresponding to four pre-selected loaded-end slip levels of the #6 and #8 test bar is shown in Figure 73 and Figure 74, respectively. As shown in these two figures, the local slip increased at each stud location as the loaded-end slip increased. There was no obvious difference of local slip subjected to a different loaded-end slip level observed in the elastic portion along the embedded bar. A substantial increase in local slip occurred in the inelastic portion along the embedded bar when the loaded-end slip increased.
Figure 73: Measured local slip distribution of a #6 test bar at 0.06, 0.08, 0.1 and 0.15 inches of loaded-end slip.

Figure 74: Measured local slip distribution of the #8 test bar at 0.1, 0.2, 0.3 and 0.4 inches of loaded-end slip.
CHAPTER 5. ANALYTICAL STUDY

In this chapter, an analytical model that can predict the bar stress vs. the loaded-end slip behavior of a reinforcing deformed bar anchored in a well-confined concrete block with sufficient embedment length is examined. The analytical model incorporates three basic elements: (1) a local bond stress-slip relationship proposed by Eligehausen (1983) with Wang’s modification factor (2008) to account for the bond strength reduction due to inelastic strains, (2) a stress-strain relationship for the steel, and (3) continuity conditions between steel and concrete. The strain distribution along the embedment length, the local slip distribution along the embedment length, and also the local bond stress vs. slip relationship with steel strains well above the yield strain of reinforcing bar are examined in this chapter.

5.1 Analytical Model of Deformed Bar Anchored in Concrete

The analytical model is based on a one-dimensional numerical solution approach of the bond problem to satisfy known steel stress or slip at the boundaries of the bar. In this approach, the bar is subdivided into a discrete number of N small elements of length \( dx \), and the bond stress, steel stress, and slip variation along the bar are calculated numerically by writing custom routines in the MATLAB using bond force equilibrium and slip compatibility equations. The MATLAB codes were written by Dr. Zhao and are used in the following data analysis. A differential element of the bar is shown in Figure 75.

\[ N = \frac{\pi d_h^2}{4} \sigma(x) \]

**Figure 75: Differential element of deformed bar anchored in concrete**

The model assumes that local slip results from the integration of steel strains while ignoring the concrete strains, and the integration of bond stress results in the changing of axial forces. At any discrete location along the bar length corresponding to certain element, the bar strain is: 
\[ \varepsilon(x) = \frac{du(x)}{dx} \]  
(5.1)

The constitutive relationship of the deformed bar is:
\[ \sigma(x) = f(\varepsilon(x)) \]  
(5.2)

Axial equilibrium of a unit segment \( dx \) of the bar provides:
\[ \frac{dN(x)}{dx} - \pi d_b \tau(x) = 0 \]  
(5.3)

Combining the bar strain equation, the constitutive relationship of the steel and the axial equilibrium equation gives:
\[ \tau(x) = \frac{d_b}{4} \times \frac{d}{dx} \left\{ f \left[ \frac{du(x)}{dx} \right] \right\} \]  
(5.4)

The nonlinear relationship between bar slip and local bond stress has the form
\[ \tau(x) = g(u(x)) \]  
(5.5)

Therefore, the governing differential equation, expressed in terms of slip, is
\[ g(u(x)) = \frac{d_b}{4} \times \frac{d}{dx} \left\{ f \left[ \frac{du(x)}{dx} \right] \right\} \]  
(5.6)

where \( \varepsilon(x) \) and \( \sigma(x) \) are the strain and stress of steel, where \( u(x) \) is the bar slip, and where \( \tau(x) \) is the local bond stress.

Based on the above discussion, it should be noticed that the bond behavior between reinforcing steel and its surrounding concrete is very sensitive to the steel stress-strain relationship, as well as to the local bond stress-slip relationship. Therefore, these two relations need to be discussed in detail.

**5.1.1 Constitutive law of steel**

The stiffness of the load-end deformation relationship is strongly influenced by the characteristics of the steel’s stress-strain curve, especially in the post-yield region.

Therefore, an accurate stress-strain relationship is required for reinforcing steel to obtain an
accurate analytical model. The tensile stress-strain curve for monotonic loading of Grade 60 reinforcing bar used in the analytical study is shown in Figure 76.

![Tensile stress-strain curve for monotonic loading of Grade 60 reinforcement](image)

**Figure 76: Tensile stress-strain curve for monotonic loading of Grade 60 reinforcement (Priestley et al. 1996)**

This relationship could be represented by the following equations:

\[ E_s = 29000 \text{ ksi} \]

\[ f_s = E_s \times \varepsilon_s, \text{ when } \varepsilon_s \leq \varepsilon_y \]

\[ f_s = f_y, \text{ when } \varepsilon_y < \varepsilon_s \leq 0.008 \]

\[ f_s = f_y \times \left[ 1.5 - 0.5 \left( \frac{0.12 - \varepsilon_s}{0.112} \right)^2 \right], \text{ when } 0.008 < \varepsilon_s \leq 0.12 \]

In order to validate the stress vs. strain model of steel, tension tests were conducted on 12 in. Grad 60 A706 sample #6 and #8 deformed bars. During the test, an extensometer (used to measure strains) was put on the sample deformed bar and was removed at 0.025 in/in strain, which allowed the strain to be obtained directly from the tension test before the strain reached to 0.025 in/in. After the strain passed the 0.025 in/in, the tension force was continued to be applied until the sample deformed bar was ruptured, while the strain stayed at 0.025 in/in. The bar stress was calculated as the load divided by the nominal cross-sectional
area of deformed bar. The bar area reduction due to Poisson’s effect during the tension test was not considered. Figure 77 presents the stress-strain relationship comparisons between experimental results, as well as the stress vs. strain relationship used in the analytical model. Compared to the experimental stress vs. strain relationship, the steel model could satisfactorily represent the actual stress-strain behavior experienced by the deformed bar when subjected to monotonic increasing tension force.

(a) #6 deformed bar
Figure 77: Stress-strain relationship comparisons between experimental results and constitutive law used for steel

5.1.2 Local bond stress-slip relationship

As previously noted, yielding of the deformed bar could significantly reduce the bond stress due to the reduced contact area between the deformed bar and surrounding concrete and introducing a modification factor is accepted as an effective way to account for the bond strength reduction due to inelastic strains (Lowes, 1999). The modification factor proposed by Wang (2008) was shown to improve the bar stress vs. the loaded-end slip behavior of embedded deformed bar (Wang, 2008). This modification factor was deemed as a function of inelastic strain, which could be expressed as:

$$\epsilon = 1 - 0.5 \left( 1 - e^{0.5 \left( 1 - \frac{\epsilon}{\epsilon_y} \right)} \right)$$

(5.7)

where, $\epsilon$ is the steel strain and $\epsilon_y$ is the yield strain of steel. This modification factor is applied to the well-recognized local bond stress-slip model proposed by Eligehausen (1983) as a local bond stress multiplier to get the complete local bond-slip model.
5.2 Boundary Conditions for this Analysis

The boundary conditions at two ends of the deformed bar are given as stated below. The slip at the free end of the deformed bar could be measured directly by the DCDT attached to the free end during the pullout tests. Based on the results from the pullout tests conducted on the #6 and #8 deformed bars, there was no movement occurred at the free end. This means that the embedment length was sufficient; therefore, the deformed bar could be fully developed prior to experiencing fracture.

The nonlinear boundary value problem could be solved through a shooting technique, along with the appropriate boundary conditions. The shooting technique could transform the boundary value problem into an initial value problem. The boundary condition is guessed at the loaded end. By doing integration along the embedded length, the slip and/or bar stress at the free end should meet certain criteria. Otherwise, a new simulation round should be conducted. In our case, since the analytical study is conducted to examine the modification factor that accounts for bond strength reduction in inelastic strains proposed by Wang (2008), the anchored deformed bar is required to be pulled out well into the inelastic range. As noted in the section 3.7, the anchored deformed bar was subjected to loaded-end slip control and allowed the bars to experience significant inelastic strains.

For this analysis, the entire bonded deformed bar is subdivided into 48 intervals. Because both the constitutive law of the steel and the local bond stress-slip model are nonlinear, the solution is found iteratively. For any load level with a certain pre-selected loaded-end slip, a trial value is given to the strain at the loaded end. It may be appropriate to assume that the entire deformed bar is initially strained. The bond stress, steel stress and slip are computed at each section using the bond force equilibrium equations and slip compatibility equations, starting from the loaded-end and ending at the free end. Therefore, the validity of the initial guess of the strain at the loaded end could be judged based on the boundary condition at the free end depending on the bar stresses at the free end equals to zero or not. If the strain calculation is advanced to a point, where the strain becomes negative or zero, the bond stress should approach to zero; otherwise a new round of simulation should be performed with a new trial strain value at the loaded-end.
5.3 Analytical Model Limitations

The analytical model is developed to predict the bar stress vs. loaded-end slip behavior of a deformed bar anchored in a well-confined concrete block. However, it should be noticed that the analytical model has some limitations that are due to the following reasons.

First, the bond stress (though described as continuous shear stress along the deformed bar), is the average effect of discrete bearing forces on the ribs, especially after the first slip, which breaks the initial chemical bond. Hence, the bond stress and slip calculation may be affected by the selection of positions relative to the ribs.

Second, the bond stress is calculated based on the difference of the bar stress at each increment according to the force equilibrium; therefore, the bond stress calculation depends a lot on the accuracy of the steel model, especially in the plastic range.

Third, the slip measured at the loaded-end does not represent the real slip, since a small piece of concrete block may get separated from the rest of the concrete block at the loaded-end (as observed in the pullout tests that were discussed in the experimental study). Therefore, the bond between deformed bar and surrounding concrete could be completely lost and there may not be any relative deformation between the steel and concrete in that region. The concrete cone forms at a depth of a couple of lug spacing or the entire unconfined concrete once the bond stress exceeds a critical value [Ueda et al. 1986].

Fourth, the concrete is treated as a rigid body that the concrete deformation is not included in the analytical model formulation. Also, the analytical model doesn’t account for the concrete cracking around the test bar at the loaded end.

5.4 Analytical Results

The analytical model presented in this chapter is developed to examine the modification factor proposed by Wang (2008) through comparison to the bar stress vs. loaded-end slip model proposed by Zhao and Sritharan (2007).

As stated in section 3.3, five pullout tests were completed. Test 2 (#6 deformed bar) and Test 3 (#8 deformed bar) are selected for comparison with the results obtained from this analytical study. The first simulation test was carried on a #6 deformed bar with an embedment length of 36 in. (48 bar diameter) under monotonic increasing tension force
applied at one end until the deformed bar was fractured. The second simulation test was carried on a #8 deformed bar with an embedment length of 48 in. (48 bar diameter) under monotonic increasing tension force applied at one end until the deformed bar was fractured. Same material properties as the test bars and concrete block in Test 2 and Test 3 were used in the analytical model.

The analytical results are presented below in the following four categories: (1) bar stress vs. loaded-end slip relationship, (2) the strain distribution along the embedment length with the loaded-end strain above the yield strain of deformed bar, (3) the local slip distribution along the embedment length with the loaded-end strain above the yield strain of deformed bar, and (4) the local bond stress-local slip relationship due to inelastic strains based on the modification factor proposed by Wang (2008).

5.4.1 Bar stress vs. loaded-end slip relationship

The yield and ultimate loaded-end slip comparisons among those obtained from the MATLAB analysis with modification factor included in the local bond-slip model, without modification factor included in the local bond-slip model, and the equation proposed by Zhao and Sritharan (2007) are presented in Table 13 for the #6 deformed bar and are presented in Table 14 for the #8 deformed bar. With respect to the loaded-end slip corresponding to the yield strength, the difference between the MATLAB analysis and the equation proposed by Zhao and Sritharan is relatively small, which confirms that the local bond-slip model proposed by Eligehausen could satisfactorily represent the bond slip behavior in the elastic range. However, the ultimate loaded-end slip derived from the model proposed by Zhao and Sritharan is much greater than that derived from the MATLAB analysis.

<table>
<thead>
<tr>
<th>Loaded-end slip</th>
<th>With modification factor</th>
<th>Without modification factor</th>
<th>Zhao and Sritharan’s model</th>
</tr>
</thead>
<tbody>
<tr>
<td>$s_y$</td>
<td>0.018</td>
<td>0.0177</td>
<td>0.02</td>
</tr>
<tr>
<td>$s_u$</td>
<td>0.16</td>
<td>0.095</td>
<td>0.7</td>
</tr>
</tbody>
</table>

Table 13: Loaded-end slip comparisons of #6 deformed bar
Table 14: Loaded-end slip comparisons of #8 deformed bar

<table>
<thead>
<tr>
<th>Loaded-end slip</th>
<th>With modification factor</th>
<th>Without modification factor</th>
<th>Zhao and Sritharan’s model</th>
</tr>
</thead>
<tbody>
<tr>
<td>$s_y$</td>
<td>0.0186</td>
<td>0.0184</td>
<td>0.024</td>
</tr>
<tr>
<td>$s_u$</td>
<td>0.43</td>
<td>0.23</td>
<td>0.84</td>
</tr>
</tbody>
</table>

Figure 78 and Figure 79 show the bar stress vs. the loaded-end slip relationship derived from analytical study, as well as the model proposed by Zhao and Sritharan conducted on the #6 and #8 deformed bars respectively. Based on these two figures, it was noted that the bar stress vs. the loaded-end slip relationship derived from the MATLAB analysis agreed well with the model proposed by Zhao and Sritharan in elastic range with acceptable discrepancy. The difference started to occur right after the deformed bar went past the yielding point and became greater as the loaded-end slip increased. In the inelastic range, the stiffness of the bar stress vs. loaded-end slip relationship obtained from the MATLAB analysis seemed to be constant, while the stiffness of the model developed by Zhao & Sritharan decreased as the loaded-end slip increased. The stiffness of the model proposed by Zhao and Sritharan continued to decrease up to approximately zero as the bar stress approaches the ultimate bar stress. Taking a closer look at the elastic range, the MATLAB analysis produced an ascending branch that was a little stiffer compared to the model proposed by Zhao and Sritharan, which could be primarily due to the concrete cracking around the test bar being not modeled in the analytical study. However, the model proposed by Zhao and Sritharan was basically developed based on the limited test data obtained from previous pullout tests conducted on deformed bar anchored in a well-confined concrete block with sufficient embedment length.

With respect to the modification factor, it could be noted from Figure 78 and Figure 79 that the local bond-slip model with the modification factor that accounted for the bond strength reduction in inelastic strains produced a more comparable bar stress vs. loaded-end slip relationship to the model proposed by Zhao and Sritharan. For both the #6 and #8 deformed bars, the capacity of the bond between the deformed bar and the surrounding
concrete under the MATLAB analytical analysis without the modification factor was overestimated, compared to that obtained from the model proposed by Zhao and Sritharan. After applying the modification factor to the local bond-slip model, the capacity of the bond decreased and got closer to that given by the model developed by Zhao and Sritharan. Therefore, it could be concluded that the modification factor could improve the bas stress vs. loaded-end slip relationship and thus indicated that the modification factor could partially reflect the bond stress reduction after the deformed bar passes the yielding point.

Figure 78: Bar stress vs. loaded-end slip relationship comparison of #6 test bar
Figure 79: Bar stress vs. loaded-end slip relationship comparison of #8 test bar

5.4.2 Strain distribution along the embedment length

The embedment length of the deformed bar accounted for 48 bar diameter. The entire embedded length of the deformed bar was divided into 48 intervals with 1 in. increments for the #8 deformed bar and 0.75 in. increments for the #6 deformed bar. Six pre-selected loaded-end slip levels corresponding to the loaded-end strains beyond the yield strain of the deformed bar were selected to represent the strain distributions along the embedment length. The bar stress with respect to each loaded-end slip of both the #6 and #8 deformed bar is presented in Table 15.
Table 15: Bar stress with respect to loaded-end slip of deformed bar

<table>
<thead>
<tr>
<th>#6 deformed bar</th>
<th>#8 deformed bar</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loaded-end slip (in)</td>
<td>Bar stress (ksi)</td>
</tr>
<tr>
<td>0.02</td>
<td>78.77</td>
</tr>
<tr>
<td>0.04</td>
<td>85.15</td>
</tr>
<tr>
<td>0.06</td>
<td>88.01</td>
</tr>
<tr>
<td>0.08</td>
<td>90.45</td>
</tr>
<tr>
<td>0.1</td>
<td>92.21</td>
</tr>
<tr>
<td>0.15</td>
<td>96.45</td>
</tr>
</tbody>
</table>

The analytical strain distribution curve developed from the local bond-slip model with the modification factor of the #6 and #8 deformed bar is presented in Figure 80 and Figure 81, respectively. At these loaded-end slip levels, the deformed bar had already been strained beyond the yielding point for both the #6 and #8 deformed bar. The horizontal axis “position” indicated the anchorage length from the free end. Based on these two figures, it could be easily noted that the strain distribution along the embedment length could be deemed as two linear straight lines as a function of the embedment length from the free end. The intersection of the two linear straight lines was located right where the deformed bar reached the yielding point. Also, it could be noted that the loaded-end strain increased as the loaded-end slip increased and significant strain increase occurred after the deformed bar went past the yielding point. The anchorage length strained beyond the yielding point also increased as the loaded-end slip increased.
Figure 80: Analytical strain distribution developed from local bond-slip model with modification factor of #6 deformed bar.

Figure 81: Analytical strain distribution developed from local bond-slip model with modification factor of #8 deformed bar.
Figure 82 shows the analytical strain distribution comparison along the embedment length of the #6 deformed bar at four pre-selected loaded-end slip levels (i.e., 0.02, 0.04, 0.06 and 0.08 inches of loaded-end slip) between that derived from the MATLAB analytical analysis conducted with and without the modification factor included in the local bond-slip model. Figure 83 shows the analytical strain distribution comparisons along the embedment length of the #8 deformed bar at four pre-selected loaded-end slip levels (i.e., 0.04, 0.08, 0.1 and 0.2 inches loaded-end slip). Along the embedment length, the strain distribution curve was the same before the strain reached the yield strain for those obtained from the model with and without modification factor. The difference was appeared to have occurred as the deformed bar went past the yielding point, which should be expected because the modification factor was applied to the local bond-slip model in the inelastic range only. The loaded-end strain corresponding to the same loaded-end slip level obtained from the model with modification factor was smaller than that obtained from the model without modification factor along the embedment length after the deformed bar went past the yielding point. This observation confirmed that the modification factor could represent the bond strength reduction due to inelastic strains. The difference of strain distribution curve between the analytical model with and without modification factor became evident as the loaded-end slip increased. Taking a closer look at the strain distribution comparisons with respect to 0.06 and 0.08 loaded-end slip levels of #6 deformed bar, especially the portion after the deformed bar went past the yielding point, it was noticed that the strain distribution obtained from the model with modification factor has a little larger anchorage length strained beyond the yielding point than those obtained from the model without modification factor under the same loaded-end slip. The same and more obvious trend could also be observed in the #8 deformed bar.
(a) At 0.02 inches of loaded-end slip

(b) At 0.04 inches of loaded-end slip
At 0.06 inches of loaded-end slip

(d) At 0.08 inches of loaded-end slip

Figure 82: Analytical strain distribution comparisons of #6 deformed bar corresponding to 0.02, 0.04, 0.06 and 0.08 inches of loaded-end slip
(a) At 0.04 inches of loaded-end slip

(b) At 0.08 inches of loaded-end slip
At 0.1 inches of loaded-end slip

Figure 83: Analytical strain distribution comparisons of #8 deformed bar corresponding to 0.04, 0.08, 0.1 and 0.2 inches of loaded-end slip

At 0.2 inches of loaded-end slip
For the #6 deformed bar having the modification factor included in the local bond-slip model, at 0.06 loaded-end slip level with a corresponding loaded-end strain of 0.032in/in, around 18.75 inches of the 36 anchorage length, or 52.1% of the anchorage length was strained with around 4 inches of the deformed bar strained beyond the yielding point at the loaded end. The loaded-end strain, the anchorage length strained, as well as the anchorage length being strained beyond the yielding point with respect to the other three loaded-end slip levels of the #6 deformed bar are presented in Table 16. For the #8 deformed bar having the modification factor included in the local bond-slip model, at 0.08 loaded-end slip level with a corresponding loaded-end strain of 0.034in/in, around 23 inches of the 48 anchorage length, or 47.9% of the anchorage length was strained with around 4 inches of the deformed bar at loaded end strained beyond the yielding point. The loaded-end strain, the anchorage length strained, as well as the anchorage length being strained beyond the yielding point with respect to the other three loaded-end slips levels of the #8 deformed bar are presented in Table 17. Based on the results, it could be concluded that: as the loaded-end slip increased, the corresponding loaded-end strain increased; the length of the deformed bar being strained increased; and the length of deformed bar strained beyond yielding point also increased, correspondingly. The length of the deformed bar being strained beyond the yielding strain propagated further inside the specimen as the loaded-end slip increased. The length strained beyond yield was continuously increasing and therefore the part of the deformed bar that remained in elastic range was diminished. In addition, significant strain increased right after the deformed bar reached the yielding point.
Table 16: Loaded-end strain, anchorage length strained and anchorage length being strained beyond the yielding point of the #6 deformed bar with modification factor included in the local bond-slip model

<table>
<thead>
<tr>
<th>Loaded-end slip (in)</th>
<th>Loaded-end strain (in/in)</th>
<th>Anchorage length being strained (in) (% of the entire anchorage length)</th>
<th>Anchorage length being strained beyond the yielding point (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.02</td>
<td>0.0095</td>
<td>16.5 (45.8%)</td>
<td>1</td>
</tr>
<tr>
<td>0.04</td>
<td>0.025</td>
<td>18 (50%)</td>
<td>3</td>
</tr>
<tr>
<td>0.06</td>
<td>0.032</td>
<td>18.75 (52.1%)</td>
<td>4</td>
</tr>
<tr>
<td>0.08</td>
<td>0.038</td>
<td>19.5 (54.2%)</td>
<td>4</td>
</tr>
</tbody>
</table>

Table 17: Loaded-end strain, anchorage length strained and anchorage length being strained beyond the yielding point of the #8 deformed bar with modification factor included in the local bond-slip model

<table>
<thead>
<tr>
<th>Loaded-end slip (in)</th>
<th>Loaded-end strain (in/in)</th>
<th>Anchorage length being strained (in) (% of the entire anchorage length)</th>
<th>Anchorage length being strained beyond the yielding point (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.04</td>
<td>0.0236</td>
<td>14 (29.2%)</td>
<td>3</td>
</tr>
<tr>
<td>0.08</td>
<td>0.034</td>
<td>24 (50%)</td>
<td>4</td>
</tr>
<tr>
<td>0.1</td>
<td>0.038</td>
<td>14 (29.2%)</td>
<td>5</td>
</tr>
<tr>
<td>0.2</td>
<td>0.053</td>
<td>27 (56.25%)</td>
<td>7</td>
</tr>
</tbody>
</table>

Figure 84 shows the strain distribution comparison between the analytical study with modification factor included in the local bond-slip model and the pullout test results along the embedment length of the #6 test deformed bar. Based on the Figure 84 (a), (b), (c), and (d) with respect to 0.06, 0.08, 0.1 and 0.15 inches loaded-end slip, it could be noted that the strain calculated from the analytical model generally corresponded with the test results. In the elastic range, the strain obtained from the pullout test results at 27 in. embedment length from the free end was higher than that obtained from the analytical study. After the test bar
reached the yielding point, the loaded-end strain obtained from the pullout test was a little lower than that obtained from the analytical study.

(a) At 0.06 inches of loaded-end slip

(b) At 0.08 inches of loaded-end slip
Figure 84: Strain distribution comparison between analytical study with a modification factor and test results of the #6 test bar. (c) At 0.1 inches of loaded-end slip. (d) At 0.15 inches of loaded-end slip.
Figure 85 shows the strain distribution comparison between the analytical study with the modification factor and the pullout test results along the embedment length of the #8 test bar. Based on the Figure 85 (a), (b), (c), and (d) with respect to 0.1, 0.2, 0.3 and 0.4 loaded-end slip, it could be noted that the strain calculated from the analytical analysis generally agrees well with the test results in the elastic range, except for the strain measured at 36 in. of embedment length from the free end with respect to 0.4 loaded-end slip. Similar to that observed for the comparisons of the #6 test bar, the loaded-end strain obtained from the pullout test is lower than that obtained from the analytical study.

(a) At 0.1 inches of loaded-end slip
(b) At 0.2 inches of loaded-end slip

(c) At 0.3 inches of loaded-end slip
Figure 85: Strain distribution comparison between analytical study with a modification factor and test results of the #8 test bar

The experimental strain distribution along the embedment length for both the #6 and #8 test bars in elastic range agrees fairly well with that developed from the analytical study with a modification factor, and the discrepancies occur as the test bar passed the yield point. The loaded-end strains measured in the pullout tests are smaller compared to that developed from the analytical analysis. The difference partially results from the accuracy of the stress-strain relationship of steel used in the analytical analysis, especially in the plastic range. In addition, the concrete cracking near the loaded-end is not included in the analytical model. However, obvious concrete cracking at the loaded-end was observed for the #8 test bar, which could explain the more evident loaded-end strain difference noticed in Figure 85. Because of the concrete cracking around the test bar, the bond between the test bar and its surrounding concrete at the loaded-end was actually completely lost during pullout test for the #8 test bar and is partially lost for the #6 test bar. Therefore, the embedment length of the test bar in the pullout test was decreasing as the concrete started to crack at the loaded-end,
especially for the #8 test bar. Due to the reduced embedment length during the pullout test, the embedment length strained beyond the yielding point propagated much further inside the anchorage region and this situation would be magnificent as the loaded-end slip levels increased. That is believed to be the strain measured at 36 in. embedment length from the free end is much greater than that obtained from the analytical study.

5.4.3 Local slip distribution along the embedment length

Based on the analytical study conducted in this chapter, the local slip is referred to as the movement of the deformed bar relative to the concrete, which could be calculated as integration of strain distribution from the free end to the interesting point, under the assumption that there is no movement occurring at the free end. The concrete is treated as a rigid body in which the concrete strain is ignored and thereby the deformation of concrete is ignored in the analytical study. The local slip distribution corresponding to the strain distributions presented previously at each loaded-end slip level for the #6 and #8 deformed bar are shown in Figure 86 and Figure 87. In these two figures, the local slip is developed from the local bond-slip model with a modification factor. Based on these two figures, it could be noticed that a substantial increase in local slip likely occurs when the bar begins to yield. In addition, the local slip at the given location along the embedment length increases as the loaded-end slip increases.
Figure 86: Analytical local slip distribution developed from local bond-slip model with a modification factor of a #6 deformed bar.

Figure 87: Analytical local slip distribution developed from local bond-slip model with a modification factor of a #8 deformed bar.
The analytical local slip distribution comparisons between the local slip calculated based on the local bond-slip model with, and without modification factor, are presented in Figure 88 for the #6 deformed bar and are presented in Figure 89 for the #8 deformed bar. For the #6 deformed bar, the local slip distribution that was obtained from the analytical model with and without a modification factor agreed to each other pretty well. There was no obvious difference between the local slip distribution obtained from the analytical model with and without the modification factor included in the local bond-slip model. For the local slip distribution of the #8 deformed bar, with respect to the 0.2 inches loaded-end slip, it could be noticed that the local slip distribution derived from the analytical model with modification factor had an anchorage length for the deformed bar sustained elastic that was a little less than that derived from the analytical model without modification factor.

(a) At 0.02 inches of loaded-end slip
At 0.04 inches of loaded-end slip

At 0.06 inches of loaded-end slip

Matlab w/ modification factor
Matlab w/o modification factor
Figure 88: Analytical local slip distribution comparisons of a #6 deformed bar corresponding to 0.02, 0.04, 0.06 and 0.08 inches of loaded-end slip
(b) At 0.08 inches of loaded-end slip

(c) At 0.1 inches of loaded-end slip
Figure 89: Analytical local slip distribution comparisons of a #8 deformed bar corresponding to 0.04, 0.08, 0.1 and 0.2 inches of loaded-end slip

The embedment length that is disturbed with respect to each loaded-end slip level with modification factor included in the local bond stress-slip model of the #6 and #8 deformed bars are presented in Table 18 and Table 19. These agree well with what has been observed in the strain distribution diagrams. From these two tables, the disturbed length is found to increase with the increase in the loaded-end slip.

**Table 18: Disturbed length of #6 deformed bar with respect to each loaded-end slip**

<table>
<thead>
<tr>
<th>Loaded-end slip</th>
<th>Disturbed length (% the entire anchorage length)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.02</td>
<td>16.5 (45.8%)</td>
</tr>
<tr>
<td>0.04</td>
<td>18 (50%)</td>
</tr>
<tr>
<td>0.06</td>
<td>18.75 (52.1%)</td>
</tr>
<tr>
<td>0.08</td>
<td>19.5 (54.2%)</td>
</tr>
</tbody>
</table>
Table 19: Disturbed length of a #8 deformed bar with respect to each loaded-end slip

<table>
<thead>
<tr>
<th>Loaded-end slip</th>
<th>Disturbed length (% the entire anchorage length)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.04</td>
<td>14 (29.2%)</td>
</tr>
<tr>
<td>0.08</td>
<td>24 (50%)</td>
</tr>
<tr>
<td>0.1</td>
<td>14 (29.2%)</td>
</tr>
<tr>
<td>0.2</td>
<td>27 (56.25%)</td>
</tr>
</tbody>
</table>

Figure 90 shows the local slip distribution comparison between the analytical study with the modification factor included in the local bond-slip model and the pullout test results along the embedment length of the #6 test deformed bar. Based on the Figure 90 (a), (b), (c), and (d) with respect to 0.06, 0.08, 0.1 and 0.15 inches loaded-end slip, it could be noted that the strain calculated from the analytical model was generally less than that measured in the pullout test.
At 0.08 inches of loaded-end slip

(c) At 0.1 inches of loaded-end slip
Figure 90: Local slip distribution comparison between analytical study with a modification factor and test results of the #6 test bar

Figure 91 shows the local distribution comparison between the analytical study and the pullout test results along the embedment length of the #8 test bar. Based on the Figure 91 (a), (b), (c), and (d) with respect to 0.1, 0.2, 0.3 and 0.4 inches loaded-end slip, it could be noted that the local slip calculated from the analytical model was generally less than that obtained from pullout test results.
(a) At 0.1 inches of loaded-end slip

(b) At 0.2 inches of loaded-end slip
Figure 91: Local slip distribution comparison between analytical study with a modification factor and test results of the #8 test bar.

(c) At 0.3 inches of loaded-end slip

(d) At 0.4 inches of loaded-end slip
Based on the local slip distribution comparison between the pullout tests and analytical study of both the #6 and #8 test bars, it could be noted that local slip obtained from the pullout tests generally produced a higher local slip than that given in the analytical study. This could be partially a result of the test instrumentation used to measure the local slip and strain distribution in the pullout test. In order to have the stud that was welded to the test bar move freely, a LED rod was placed around each individual stud to prevent concrete from flowing into the stud location, causing a part of the test bar being not bonded. In addition, the area of the test bar had to be reduced because of the strain gauge instrumentation and the area of the test bar continued to decrease as the test bar was pulled out due to the Poisson’s effect. However, the area and circumferential length of the bar are two of the parameters defined in the analytical study. Due to the instrumentation used to measure the strain and local slip during the pullout tests, the area of the bar at each strain gauge location is smaller than the actual nominal bar area, so as to the circumferential length at each stud location. The parameters input in the analytical model could not represent the exact rebar stress-strain response, leading to the local slip measured in the pullout test being generally greater than that obtained from the analytical study. Another obvious difference between the measured local slip and the local slip developed from the analytical study occurred when the deformed bar experienced the initial yielding along the embedment length. This difference might have come from the fact that the test bar was experiencing the complicated strain hardening period and the response may be misleading.

5.4.4 Local bond Stress vs. local slip relationship

The bond stress is assumed to be uniformly distributed along each increment, which could be expressed as $u = \frac{A_s \Delta f_s}{\pi d_b \Delta x}$, where $u$ is the local bond stress, $\Delta f_s$ is the change of bar stress over the increment, $\Delta x$, $A_s$ and $d_b$ are the nominal cross-sectional area and bar diameter, respectively. The local bond stress - slip relationship with respect to each loaded-end slip developed from the improved local bond-slip model is plotted in Figure 92 for the #6 deformed bar and is plotted in Figure 93 for the #8 deformed bar. The bond-slip model with the modification factor accounting for the bond strength reduction due to inelastic strains
follows an initial nonlinear ascending branch to maximum bond stress (1600 psi) and then has a sudden sharp decrease in bond stress as the deformed bar experiences the initial yielding. This sudden transition from the ascending branch to the descending branch corresponded with the reinforcing bar yielding. The bond stress continued to decrease as the local slips increased until the local slip reached a value of 0.02 to 0.03 inches. The bond stress increased slightly after the local slip went pass 0.03 in. The bond stress stayed at 1160 psi as the local slip was in the range of 0.05 and 0.11 inches, and then followed a linear decreasing straight line until the frictional bond stress.

Figure 92: Analytical local bond stress-slip relationship of a #6 deformed bar subjected to monotonic loading
In order to investigate the effect of inelastic strain on the local bond strength-slip relationship, the local bond stress-slip relationship developed from the analytical study with the modification factor is plotted in the same figure as that proposed by Eligehausen (Figure 94 for the #6 deformed bar and Figure 95 for the #8 deformed bar). As mentioned previously, the model proposed by Eligehausen was based on pullout tests conducted on reinforcing bars with a short embedment length; therefore, this model couldn’t reflect the change in bond characteristics after the reinforcing bar yielded. Based on these two figures, it could be noticed that the maximum bond strength is much smaller than that given by the model proposed by Eligehausen. Also, the local slip corresponding to the maximum bond strength is also smaller than that proposed by Eligehausen. The linear decreasing branch for the improved bond-slip model follows a less decreasing slope than the Eligehausen’s model. However, the bond strength plateau is located around the same range of local slip for both
relationships. Because the deformed bar gets to the ultimate strength before the bond strength arrives to the frictional bond stress, there is no information available about the frictional bond strength. It is interesting to notice that there is a sharp decrease in local bond stress as the deformed bar experiences the initial yielding. However, a local bond stress-slip relationship could be derived by averaging the local bond stress at the sharp descending part as represented by the dashed lines in Figure 94 and Figure 95, respectively. This relationship needs further investigation in the future.

![Local bond stress-slip relationship comparison of a #6 deformed bar](image)

**Figure 94:** Local bond stress-slip relationship comparison of a #6 deformed bar
Figure 95: Local bond stress-slip relationship comparison of a #8 deformed bar
CHAPTER 6. CONCLUSION AND FUTURE WORK

6.1 Conclusions from Experimental Study

The hysteretic bar stress vs. loaded-end slip model proposed by Zhao and Sritharan was based on limited test data available in the literature for deformed reinforcing bars anchored in a concrete block with sufficient embedment length. Therefore, an experimental program which could simulate the bond-slip behavior of longitudinal reinforcing bars fully anchored into confined concrete simulating the bridge joints or footings was used in this research. The strain penetration effect was simulated experimentally in this study, where large inelastic strains and significant ultimate loaded-end slips were developed. Satisfactory set of test data was collected from the pullout tests and the model proposed by Zhao and Sritharan was generally consistent with the measured response. It could be concluded that the model proposed by Zhao and Sritharan appears to satisfactorily represent the bar stress vs. loaded-end slip relationship for pullout tests conducted on a deformed reinforcing bar anchored in a well-confined concrete block with adequate embedment length. The model proposed by Zhao and Sritharan underestimated the loaded-end slip at the yield strength for both #6 and #8 test bar, but overestimated the loaded-end slip at the ultimate strength for the #6 test bar, while giving a comparable ultimate loaded-end slip for the #8 test bar. As observed in the pullout test, significant cracks formed in concrete around the #8 test bar, while little cracks occurred around the #6 deformed bar. For the #8 test bar, an evident splitting of cover concrete was observed as the test bar was pulled out and significant piece of concrete block fractured away from the rest of the concrete block. Therefore, the bond between the #8 test bar and surrounding concrete decreased continuously and was completely lost as the evident splitting cover concrete was formed. By revisiting the test data used to develop the model proposed by Zhao and Sritharan, the embedment length was typically in the range of 16-30 bar diameter, which was smaller than 48 bar diameter (the embedment length of the pullout test conducted for this thesis) and obvious concrete cracks could be observed near the loaded-end. Therefore, it could be concluded that the model proposed by Zhao and Sritharan was typically applicable for the test specimen with obvious concrete cracks around the test bar presented at the loaded-end.
Four strain gauges were mounted to the test bar to record the strain at unique locations along the embedment length while the test bar was pulled out. Due to the concrete cracked around the test bar at the loaded-end, the top strain gauge located at the connection interface stopped working before the test deformed bar fractured. Therefore, several LEDs were placed along the test bar beyond the interface to obtain the loaded-end strain. The strain distribution along the embedment length for strain levels above the yield strain of the reinforcing bars was measured during the pullout tests. The strain distribution increased as the loaded-end slip level increased and significant strain increase occurred right after the test bar went pass the yielding point along the embedment length.

Three studs (for the #6 test bar) and four studs (for the #8 test bar) were welded along the test bar to directly measure the local slip at pre-selected locations. Similar to the strain distribution, the local slip along the embedment length increased as the loaded-end slip level increased and a significant increase in local slip occurred after the test bar went pass the yielding point.

6.2 Conclusions from Analytical Study

The improved local bond-slip model that considers the effect of inelastic strains, and the constitutive law of steel, were incorporated in an analytical model to simulate the bar stress vs. loaded-end slip relationship of reinforcement in concrete. The predicted bar stress vs. loaded-end slip relationship was compared to the model proposed by Zhao and Sritharan (2007). The analytical results were shown in good consistency with the model proposed by Zhao and Sritharan (2007) in elastic range. The relationship obtained from the analytical study produced a little stiffer ascending branch compared to the model proposed by Zhao and Sritharan (2007), which could be primarily due to the fact that the analytical model did not include the concrete cracking at the loaded-end. The bar stress vs. loaded-end slip relationship developed from the local bond-slip model with the modification factor produced a curve more comparable to that of the model proposed by Zhao and Sritharan (2007), which indicated that the modification factor could improve the global response of a reinforcing bar anchored in a well-confined concrete block with sufficient embedment length. Therefore, it could be concluded that the modification factor could partially represent the bond stress reduction after the reinforcing bar went past the yielding point. Once the reinforcing bar
went past the yielding point, the stiffness of the relationship that was developed from the analytical study seemed to be constant, while the stiffness of Zhao and Sritharan’s model decreased as the loaded-end slip increased. The stiffness of Zhao and Sritharan’s model continued to decrease up to approximately zero as the bar stress approached the ultimate bar stress. Therefore, the ultimate loaded-end slip for Zhao and Sritharan’s model was much greater than that obtained from the analytical study. The difference could be contributed to the fact that the concrete cracking at the loaded-end was not modeled in the analytical model to simulate the load-end deformation behavior. The concrete cracking at the loaded-end would result in either the bond strength between the deformed bar and its surrounding concrete decreasing or becoming completely lost. Since the study was primarily focused on the effect of inelastic strains, the strain distribution and local slip distribution derived based on the analytical study corresponding to different levels of loaded-end slip beyond yielding were also discussed. The strain distribution along the embedment length could be deemed as two straight lines with the intersection located right at the yielding point. The strain increased significantly along the embedment length after the bar passed the yielding point. Also, the embedment length of the deformed bar strained, as well as the embedment length of the deformed bar being strained beyond the yielding point, were both increased as the loaded-end slip increased.

The strain distribution along the embedment length for both the #6 and #8 deformed bar developed from the analytical model with a modification factor included in the local bond-slip model generally agreed fairly well with the pullout test results. In the elastic range along the embedment length, the strain obtained from the pullout tests was a little greater than that obtained from the analytical study, which could be partially resulted from the reduced bar area in the pullout test, compared to the nominal bar area input in the analytical study. The reduced bar area came from the strain gauge instrumentation. After the bar strained beyond the yielding point, the experimental strain distribution along the embedment length produced lower strains than that obtained from the analytical study. This difference came partially from the accuracy of the stress-strain relationship of the steel used in the analytical study, especially in the inelastic range. In addition, since the concrete crack at the loaded-end was not included in the analytical model, the analytical study actually overestimated the bond
capacity. Once the concrete cracked at the loaded-end, the bond between the deformed bar and its surrounding concrete was lost. This would lead to the actual embedment length decreasing, which was not accounted for in the analytical model.

Similar to the strain distribution, a significant local slip also occurred after the reinforcing bar passed the yielding point. The local slip directly measured by the LED in the pulled out tests was greater than that obtained from the analytical study, especially when the interesting point moved toward the loaded-end along the embedment length. The difference came partially from the fact that the concrete crack at the loaded-end was not modeled in the analytical study. Also, the LED rod around each stud reduced the circumferential length of the bar and part of the bond strength was not obtained in the pullout tests. In addition, due to the splitting of cover concrete at the loaded-end, the local slip measured close to the loaded-end could not actually represent the local slip at that point. Although the local slip measured near the loaded-end seemed to be in consistency with the local slip derived from the analytical model for the #8 test bar, it actually represented the local slip at a position located further inside the embedment length.

Based on the analytical study, a local bond-slip relationship considering the inelastic strains was also derived. This relationship clearly represented the bond strength reduction due to inelastic strains. This relationship follows an initial nonlinear ascending branch to maximum bond stress (1600 psi) and then has a sharp decrease in bond stress as the deformed bar experiences the initial yielding. The bond stress continues to decrease as the local slips increase until the local slip arrives to around 0.02 to 0.03 in. The bond stress increases a little after the local slip passes 0.03 in. While the local slip is in the range of 0.05 and 0.11 in, the bond stress stays at 1160 psi, and then follows a linear decreasing straight line. Due to the limited information, there was no way to investigate the frictional bond strength. The deformed bar in the analytical study reached the ultimate strains before approaching the frictional bond strength.

6.3 Recommendation for Future Research

Although the analytical study and experimental study discussed in this thesis have presented the local bond-slip behavior subjected to inelastic strains, as well as compared the bar stress vs. loaded-end slip model with that proposed by Zhao and Sritharan, the model
used in the analytical study has some limitations, and further investigation will be
appropriate. A few recommendations for future research include:

(1) Future analytical study should include the concrete crack around the test bar at the
loaded-end.

(2) The embedment length reduction occurring during the test bar being pulled out should
be considered in the analytical model.

(3) The test bar was fractured inside the concrete block, which was not expected in the
analytical study.

(4) The area and circumferential length of the bar used in the analytical model should
reflect the actual situation.

(5) A more realistic steel stress vs. strain relationship needs to be incorporated into the
analytical model, especially in the plastic range.

(6) The modification factor needs further investigation.

In the past few years, prestressed concrete has become a popular structural material.
With the development of the prestressed concrete industry, untensioned bonded prestressing
strands have been used in some design especially that involves accelerated bridge
construction. With all of the new technology developed, questions arise and corresponding
testing needs to be done to determine the anchorage characteristics of strand. Similar pullout
tests are planned to be conducted in the second phase of this research.
REFERENCES


