ANALYSIS OF MECHANICALLY SPLICED TENSION CONNECTIONS

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U.S. DEPARTMENT OF THE INTERIOR
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## Analysis of Mechanically Spliced Tension Connections

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### Abstract
Strength testing in the Bureau of Reclamation laboratories of nine proprietary mechanical splicing techniques currently available in North America revealed that all exemplar No. 9 spliced specimens met the basic strength criterion of 125 percent of the specified yield. Some specimens also met additional strength and strain requirements outlined by more stringent specifications. The testing program included a series of tests to evaluate other splice properties including stiffness, slippage, and elongation. Nonspliced control bars were tested for conformance to ASTM A 615 for deformed bars. The appendix contains descriptions of the various splicing techniques, which will help the engineer select the splicing method best suited for the design application. Designers who must specify splicing systems and respond to contractor submittals in a timely fashion will find this information beneficial.

### Key Words and Document Analysis

#### a. DESCRIPTORS--
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embedded metal/ reinforcing materials/ reinforcing steels/ structural steel/

#### b. IDENTIFIERS--
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ANALYSIS OF MECHANICALLY SPLICED TENSION CONNECTIONS

by

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INTRODUCTION

The purpose of this research program at the Bureau of Reclamation was to identify, test, and compare samples of various proprietary mechanical splicing techniques currently available in North America and to evaluate the performance against some of the more stringent specifications. In addition, testing was conducted to obtain information useful in evaluating the potential limitations not commonly revealed by strength testing alone. Familiarity with the data comparisons and proprietary techniques and codes will allow designers to quickly identify and specify the splicing operation most suited to the design application.

Fabrication, manufacturing, and transportation parameters limit the use of full-length continuous steel bars in most reinforced concrete structures. Therefore, proper splicing of reinforcing steel is essential to the integrity of any reinforced concrete structure. Three methods currently available for splicing rebar include: lap splicing, weld splicing, and mechanical splicing. In general, lap splicing is the most economical and frequently chosen method.

Codes pertaining to lap splices can require the use of long laps, which cause congestion at the splice location. If the reinforcement is closely spaced, lap splicing may be virtually impossible. When large diameter bars (No. 9 and greater) are specified, the overlapping steel may also cost more than the labor and material costs of a mechanical splice using a butted rebar connection. Construction methods, the location of construction joints, and the provision for future construction can also make lap splices impractical. In addition, the American Concrete Institute Building Code for Reinforced Concrete (ACI 318-89, section R12.14.2.1) does not permit lap splices of No. 14 and 18 rebar except in compression and then only to No. 11 bars and smaller or where tension tie members are used.

If lap splices are not permitted or are impractical, welded splices or mechanical connections must be used. Welded splices require close field control to adjust welding techniques to the chemistry of the reinforcing steel and are generally the slowest, most expensive, and least reliable connections. In addition, welded splices are generally not permitted by Bureau of Reclamation specifications. Mechanical connections, therefore, are frequently the only available option. Current approval procedures, however, generally require submittal of contractor-spliced samples on a project- by-project basis. The approval process frequently involves extensive laboratory testing and analysis. Because contractors often request alternative splicing systems when construction is well underway, the approval process may result in untimely delays. Information pertaining to the limitations and performance of the splicing systems is therefore beneficial and necessary to designers who must specify splicing systems and respond to contractor submittals in a timely fashion.

Eleven different tension connection techniques were identified. Spliced and nonspliced samples consisting of No. 9, grade 60, rebar were obtained from seven different manufacturers representing nine of the proprietary techniques. Spliced specimens and corresponding control bars were fabricated from the same heat of steel. The proprietary techniques were categorized using the descriptive terminology of Mechanical Connections of Reinforcing Bars (ACI 439.3R) and listed in conjunction with their identification label and manufacturer. Techniques tested included:

1. Cold-swaged steel coupling sleeve – DBGC specimens (company 3).
2. Extruded steel coupling sleeve – DEC specimens (company 2).

3. Cold-swaged steel coupling sleeves with threaded ends acting as a coupler – DBGTC specimens (company 3).


5. Coupler for thread-deformed reinforcing bars – DLNC specimens (company 2).

6. Taper-threaded steel coupler – LC and FHC specimens (companies 5 and 7, respectively).

7. Threaded couplers with standard national coarse threads – WFC specimens (company 1).

8. Forged threaded steel coupler with standard rolled threads (dowel bar mechanical connection systems) – RC specimens (company 6).


The untested proprietary techniques included the grout-filled coupling sleeve and a new technique simply described as a steel coupling sleeve with a wedge. A description of the proprietary mechanical splicing technique associated with the test specimen abbreviations and company may be found in the appendix.

Samples from all aforementioned techniques were tested for conformance to basic criterion which required strengths of at least 125 percent of the specified yield (75,000 lb/in²) of an unspliced rebar. ACI 318-89 (section 12.14.3.4) and the 1984 AASHTO Standard Specifications for Highway Bridges are two of several specifications that outline these requirements. In addition, the samples labeled DBCG, SC, FHC, RC, and LCT were identified by product manufacturers as capable of achieving 100 percent of the minimum specified tensile strength of the reinforcing steel (90,000 lb/in²). Specimens labeled WFC and DEC were also identified as capable of meeting this requirement under certain conditions, which generally required upscaling the threaded rebar to the next larger size or specifying alternate splicing details. These conditions, however, were not specified prior to sample submittal in these cases.

While design codes generally contain criteria for basic strength requirements, the most frequently specified and less stringent codes do not contain criteria for evaluating other potential limitations. Testing was therefore conducted to evaluate the performance of mechanical splices against some of the more stringent specifications including:


4. ACI 349-85 Code Requirements for Nuclear Safety Related Concrete Structures – conformance to cyclic criteria in Section 14.14.3.4(b) was not tested.
5. ACI 359-86 Code Requirements for Concrete Reactor Vessels and Containments.

Special considerations such as those primarily encountered in seismic design (i.e., coupler resistance to fatigue, stress reversal, dynamic load, and longtime creep) were beyond the scope of this research. Test data in these cases should be secured directly from the manufacturer or other reputable sources.

CONCLUSIONS

When subjected to testing, all spliced specimens achieved tensile strengths greater than 75,000 lb/in², therefore meeting the basic strength criterion of 125 percent of the specified yield.

Mechanical connections were also tested for conformance to more stringent strength and strain requirements of the specifications mentioned above. All of the exemplar specimens achieved ultimate tensile strengths in excess of 80,000 lb/in², therefore meeting the State of California Department of Transportation criterion. In addition, all spliced specimens also achieved tensile strengths greater than 0.9f Y (81,000 lb/in²), therefore meeting United States Corps of Engineers and the State of Washington Department of Transportation requirements. Although the cyclic criterion of ACI 349-85 was beyond the scope of this research, computations for strain criteria indicated the need to stagger spliced specimens from the DLN and LCT series if the computed design stress exceeded 0.5f Y (30,000 lb/in²).

Testing for ACI 359-86 conformance indicated that specimens WFC-2, DLNC-1, DBGTC-1, LCT-1, and LCC-1 did not achieve 90 percent of the control bar strength while specimens WFC-2, FH-1, and LCC-1 failed to achieve the minimum required tensile strength of 90,000 lb/in². Since 90 percent of the control bar strength exceeded 1.25f Y (75,000 lb/in²) in all cases, the manufacturers did not design specimens WFC-2, DLNC-1, DBGTC-2, and LCC-1 to obtain such strengths. Although sample LCT-1 was designed to achieve strengths exceeding 90,000 lb/in², and did so, it failed to meet the criterion requiring a splice strength greater than 90 percent of the control bar. Sample FH-1 also failed to meet ACI 359-86 criterion requiring a splice strength greater than 90,000 lb/in². The control bar strength of sample FH-1, however, was much lower than that of the other exemplar bars. If the FH-1 control bar strength had approached 100,000 lb/in² rather than 90,000 lb/in², data comparisons indicated that it would have also likely passed the minimum tensile strength requirement.

When expressing splice strength as a percentage of control bar strength, none of the proprietary connections achieved ultimate strengths equivalent to that of the corresponding control bars. Instead, splice strengths ranged from 80.9 percent to 97.6 percent of the control bar strength.

Strength testing alone cannot always reveal the potential limitations of splicing systems, so another objective of the testing program was to obtain information useful in evaluating these limitations. Four series of tests were conducted. The first test series analyzed design assumptions by comparing reinforcing steel properties with connecting splice properties under equivalent loading conditions. The testing showed that in most cases, yielding of the reinforcing steel occurred simultaneously with that of the connecting splice, indicating that the connection behaved as a continuous unspliced bar. In a few cases, however, yielding occurred across the connecting splice first, followed by yielding of the reinforcing steel (specimens DLNC-1, LCT-1, and LCC-1). In one of these cases (DLNC-1), the failure occurred through the splice. In the other two cases, the failure occurred in the reinforcing bars. Even though the specimens
obtained specified strengths, these results indicated that the splice reduced the longitudinal stiffness in such cases.

The second and third test series were conducted to determine the slippage potential of the reinforcing steel within the splice. Slippage can cause large, unanticipated deflections and detrimental concrete cracking in reinforced concrete structures, which can in turn compromise the corrosion protection of the reinforcing steel and increase the potential for premature deterioration. Testing showed that the total slip exceeded the maximum permitted value of 0.010 inch in three cases when specimens were loaded to 50 percent of the specified yield and relaxed to 3,000 lb/in$^2$ (specimens DLNC-2, DLNC-3, and WFC-4). The total slip, however, never exceeded the maximum permitted value of 0.045 inch in any case when specimens were loaded to 90 percent of the specified yield and relaxed to 3,000 lb/in$^2$.

The third test series also evaluated the longitudinal ductility (elongation) of the spliced specimens, which is critical to the seismic performance of reinforced concrete structures. Elongation measured across the reinforcing steel was usually double, and in a few cases five times greater than, the elongation measured across the rebar-coupler interface. Elongation measured across the coupler itself ranged from 0 to 3.3 percent. Although the initial gauge lengths varied in proportion to the size of the splice, these values suggested that the elongation of spliced specimens restrained by the splice will be less than that of a continuous unspliced bar in most cases.

The fourth test series was performed to verify conformance of nonspliced specimens to the requirements of Deformed and Plain Billet-steel Bars for Concrete Reinforcement (ASTM A 615) for deformed bars. All exemplar specimens met the following criteria: minimum tensile strength of 90,000 lb/in$^2$, minimum yield of 60,000 lb/in$^2$, and minimum elongation of 7 percent over an 8-inch gauge length.

**TEST SPECIMENS**

The cost and complexity involved in instrumenting, testing, and obtaining reinforcing steel of all sizes from various manufacturers dictated the choice of one representative sample size, No. 9 rebar, for all testing. Samples representing most proprietary mechanical splicing techniques available for use in North America, as well as nonspliced control specimens, were obtained from a variety of manufacturers. Samples as received from each individual manufacturer were fabricated from the same heat of steel and complied with ASTM A 615 for grade 60 rebar. The manufacturers prepared spliced specimens prior to shipping. The splices can be considered representative of those properly performed in the field because the same procedures are used. The only tension splicing methods not tested were the grout-filled coupling sleeve and the steel coupling sleeve with a wedge because of the difficulty encountered in obtaining test specimens.

**TESTING PROCEDURE**

Only the engineer familiar with overall structural design, probable construction conditions, final conditions of service, and the unique requirements of proprietary mechanical splicing techniques can properly evaluate the variables to select the most efficient and economical splicing method. To help the engineer make a selection and interpret the test results contained
in this report, descriptions of the various proprietary mechanical splicing techniques are in order. The descriptions, which appear in the appendix, outline the splice method, the preparation of bar ends, the spacing and clearance considerations, and any other special considerations unique to the individual splicing operation. Many of these tension splicing systems also satisfy compression splice requirements.

**Design Assumptions**

To ensure the structural integrity of reinforced concrete structures, construction materials must meet certain design code criteria. Designers also make assumptions regarding the performance of such materials to allow effective use of design methodologies. When using mechanical connections, designers commonly assume that:

- Spliced reinforcing steel will act as one continuous unspliced bar.
- The connection will not introduce a weakness detrimental to the overall structural integrity.
- The connection will not reduce longitudinal stiffness and ductility.

Because code requirements commonly specify that mechanical connections must achieve a minimum tensile strength of at least 125 percent of the specified yield of the bar, designers also assume that yielding will occur in the reinforcing bar adjacent to the mechanical connection prior to failure of the connection itself.

**Testing Objectives**

Although design and construction specifications generally contain criteria for basic strength requirements, the most frequently specified and less stringent specifications do not contain criteria for evaluating other potential limitations. A testing program was therefore developed to analyze design assumptions and to compare and evaluate the performance of various mechanical splices against some of the more stringent specifications requirements currently used for mechanical splices. The four series of performance tests involved destructive static tensile tests of three spliced specimens and one unspliced specimen from each manufacturer. Each series of tests was performed to obtain information on the potential limitations of splicing systems not commonly obtained by strength testing alone. The test objective will be discussed first, followed by a description of the test procedure and a summary of the information obtained from the test series.

**First Test Series**

In a flexural member, the mechanical connection must not reduce the effective longitudinal stiffness of the reinforcing steel so as to violate the strain conditions assumed in the design. Designers commonly assume a balanced strain condition in which the strain in the tension reinforcement will exceed or equal the yield strain when the concrete strain reaches ultimate (i.e., the actual reinforcement ratio does not exceed the effective balanced design ratio). A reinforcement ratio greater than the effective balanced design ratio will cause a decrease in flexural strength. Because balanced strain conditions depend on the modulus of elasticity of the reinforcement in flexural members, the coupler material cannot be so soft (have such a low effective modulus) that this design criterion is violated. The first test series was therefore conducted on spliced specimens to compare reinforcing steel properties with connecting splice
properties under the same test conditions. Designers may find such information useful in evaluating strain conditions unique to particular designs.

Spliced specimens were instrumented with two strain gages at the coupler midpoint and on the rebar 4 inches away from one coupler end. Steel plates were tack-welded onto the coupler ends and three linear variable displacement transformers (LVDT's) were fixed between these reference points to measure strain across the splice. Readings from three LVDT's and four strain gages were averaged to minimize the effect of potential eccentric loading conditions at the coupler (caused by splicing unaligned rebar). Strain gage readings were measured until strain gage failure, which typically occurred shortly after specimen failure. The LVDT's, however, were usually removed after a minimum coupler elongation of 2 percent to prevent damage caused by shock loads at failure. Load-strain curves (figs. 1-22) were generated as specimens were tested in tension to failure, providing comparisons of properties at the rebar, at the coupler midpoint, and across the coupler.

Each load-strain curve for the first test series provided the following information (see table 1):

1. Tensile strength of the spliced specimen.
2. Yield at coupler midpoint, across the coupler, or on the reinforcing steel 4 inches away from one coupler end.
3. Chord modulus of elasticity of the reinforcing steel.

The failure mode of the spliced specimen was also noted.

Second Test Series

To ensure the structural integrity of reinforced concrete structures, conventional design loads must not produce large, unanticipated deflections causing detrimental concrete cracking. Such cracking compromises the corrosion protection of the reinforcing steel and increases the potential for structural deterioration. Splicing methods that minimize slippage are therefore desirable. The second test series was conducted on spliced specimens to determine slippage of the reinforcing steel within the coupler.

Steel plates were tack-welded to the rebar 2 inches away from each coupler end. Readings from three LVDT's were averaged to measure strain between the fixed reference points across the rebar-coupler interface. Specimens were cycled in tension to 50 percent of the specified yield and relaxed to 3,000 lb/in², then cycled again to 90 percent of the specified yield and relaxed to 3,000 lb/in². The deformation at 3,000 lb/in² was considered indicative of coupler slippage. Load-deformation curves (figs. 23-33) were produced. Test samples were not taken to failure so couplers could be cut in half and visually inspected.

Each load-deformation curve for the second test series provided the following information (see table 2):

1. Slip after cycling to 50 percent of the minimum specified yield of the reinforcing steel.
2. Slip after cycling to 90 percent of the minimum specified yield of the reinforcing steel.
Slippage was also confirmed visually as the splice was cut in half and examined.

**Third Test Series**

In structures subject to seismic excitation, potential load reversals, and potential inelastic straining, the ductility of mechanical connections must be such that failure initiates in the concrete rather than the steel reinforcement. Mechanical splicing techniques that use threads are more susceptible to notch effects in such cases. Notch effects, if present, are generally introduced in the threaded portion of the reinforcing steel at the splice end. Notch effects induce localized inelastic straining, prevent the spread of bar yielding to the adjoining bar stock, and may contribute to premature failure of the mechanical connection. Dynamic or fatigue loadings and cold temperatures also actuate notch effects. Although dynamic and fatigue testing of mechanical connections was beyond the scope of this study, a third test series examined the longitudinal ductility or elongation of the spliced connections. To maximize data collection, the slippage of the reinforcing steel within the coupler was also determined.

Steel plates were tack-welded to the rebar 2 inches from each coupler end and readings from three LVDT's were averaged to measure strain between the fixed reference points across the rebar-coupler interface. Gauge marks were placed over the centerpoints of both abutting rebars in each splice as well as across the coupler-rebar interface and on the coupler itself to allow manual measurement of elongation after bar failure. Specimens were cycled in tension to 50 percent of the specified yield and relaxed to 3,000 lb/in², then cycled to 90 percent of the specified yield and relaxed to 3,000 lb/in². The specimens were then taken to failure. The deformation at 3,000 lb/in² was once again considered indicative of coupler slippage. The LVDT's were usually removed after a minimum coupler elongation of 2 percent to prevent damage caused by shock loads at failure. Load-deformation curves (figs. 34-44) were produced to aid in material property determination and data comparisons.

Each load-deformation curve for the third test series provided the following information (see table 3):

1. Tensile strength of the spliced specimen.
2. Slip after cycling to 50 percent of the minimum specified yield of the reinforcing steel.
3. Slip after cycling to 90 percent of the minimum specified yield of the reinforcing steel.
4. Elongation of reinforcing steel, coupler, and coupler-rebar interface.

The failure mode of spliced specimen was also noted.

**Fourth Test Series**

A fourth test series was performed on the nonspliced reinforcing steel to confirm that nonspliced specimens met the requirements of ASTM A 615. The testing also permitted data comparisons between spliced and nonspliced specimens. Gauge marks were placed over the centerpoints of the rebar to allow manual measurement of elongation after bar failure. Specimens were then instrumented with strain gages and loaded to failure in tension. Because all samples (spliced and nonspliced) were fabricated from the same heat of steel, the bar...
was tested from each group submitted by various manufacturers. Stress-strain curves were generated (figs. 45-53).

Each stress-strain curve for the fourth test series provided the following information (see table 4):

1. Tensile strength.
2. Yield.
3. Elongation over an 8-inch gauge length.

DATA CALCULATION AND SPECIFICATIONS REQUIREMENTS

Ultimate strength was calculated by dividing ultimate measured load by the area of a standard No. 9 bar (1.0 in²) for both spliced and nonspliced specimens. ASTM A 615 requires that grade 60 deformed bars achieve a minimum ultimate tensile strength of at least 90,000 lb/in². Most construction specifications also require mechanical connections to meet the basic strength requirements of ACI 318-89, section 12.14.3.4, which states, "A full mechanical connection shall develop in tension or compression, as required, at least 125 percent of specified yield strength (f_y) of the bar." For grade 60 steel, the mechanical connection must therefore achieve an ultimate tensile strength of at least 75,000 lb/in².

In addition to these basic strength requirements, more stringent specifications may also be referenced as the situation dictates or when a higher standard of performance is desired. For instance, in addition to the basic strength criterion, the mechanical splice specifications of ACI 349-85 for Nuclear Safety Related Concrete Structures also impose strain criteria. Section 12.14.3.7 states, "Mechanical connections shall be staggered if the strain measured over the full length of connector (at 0.9 yield) exceeds that of an unspliced bar by more than 50 percent and if the maximum computed design load stress in the bar equals or exceeds 0.5f_y." In other words, if bars are loaded to a computed design stress in excess of 30,000 lb/in², the splices must be staggered if the strain across the splice of exemplar specimens loaded to 54,000 lb/in² is greater than 1.5 times the strain of corresponding nonspliced control bars at 54,000 lb/in². These previous stress values apply only to grade 60 rebar.

ACI 349-85 section 12.14.3.4(b) also imposes cyclic criteria requiring exemplar specimens to be subject to 100 cycles of tensile stress varying from 5 to 90 percent of the specified minimum yield strength of the reinforcing bar. The specimens must withstand the cyclic testing without loss of static tensile strength capacity when compared to like specimens subjected to noncyclic failure and tested statically to failure following cyclic tests. As previously mentioned, cyclic testing was beyond the scope of this research and is mentioned here only for completeness.

The mechanical splice requirements of ACI 359-86 for Concrete Reactor Vessels and Containments state that the average tensile strength of the splice shall not be less than 90 percent of the actual tensile strength of the reinforcing bar being tested, nor less than 100 percent of the specified minimum tensile strength (90,000 lb/in² for grade 60 rebar). The United States Corps of Engineers’ Civil Works Construction Guide Specification and the
Standard Specification for Road and Bridge Construction of the State of Washington Department of Transportation also make similar requirements. However, they specify that splices must develop a strength of at least 90 percent of the specified minimum ultimate tensile strength of the bars (81,000 lb/in\(^2\) for grade 60 rebar) as opposed to 90 percent of control bar strength. The State of California Department of Transportation requires a minimum splice strength of 80,000 lb/in\(^2\).

In addition to strength determinations, several other rebar properties of the instrumented specimens were evaluated. The resulting curves and data printouts provided the yield point of the instrumented specimens. The yield point, as defined by Mechanical Testing of Steel Products (ASTM A 370), is the first stress less than the maximum obtainable stress at which strain increases without an increase in stress. A stress-strain curve may also define yield point as the maximum stress corresponding to the top of the knee on the first curve. In addition, the load indicator of the testing machine often pauses at the yield point during the test. When the reinforcing steel was instrumented on the bar, all three methods were used to ensure an accurate estimate of the yield point. ASTM A 615 requires that deformed bars achieve a minimum yield of at least 60,000 lb/in\(^2\).

For spliced specimens instrumented at the coupler midpoint or across the coupler itself, the yield point could not be characterized by an obvious disproportionate deformation easily measured by the pause of the load indicator or the autographic methods previously described. Therefore, the yield was determined by the total-extension-under-load method. As a specimen is loaded to obtain a specified extension, the load or stress corresponding to the specified extension is recorded as the yield point. For nonspliced steel with a yield strength under 80,000 lb/in\(^2\), an appropriate value of 0.005 in/in of gauge length is usually assumed. The yield point was therefore estimated as the load corresponding to 0.005 in/in of strain (5000 microstrain) on the load-strain curve.

The modulus of elasticity was calculated as the chord connecting two specified points within the elastic range on the stress-strain curve. The upper stress point was set at 35,000 lb/in\(^2\), while the lower stress point was set at 5,000 lb/in\(^2\). The modulus of elasticity for conventional reinforcing steel generally ranges between 26 \(\times\) 10\(^6\) and 32 \(\times\) 10\(^6\) lb/in\(^2\).

Elongation was determined by piecing together the two broken halves of the reinforcing steel and measuring the distance between the gauge marks spanning the fracture. Elongation was calculated by subtracting the initial gauge length from the measured distance recorded after fracture and dividing the result by the initial gauge length and multiplying the quotient by 100. As required by ASTM A 615 for conventional reinforcing steel, the initial gauge length is standardized at 8 inches. An elongation greater than or equal to 7 percent for No. 9 bars is considered acceptable. Elongation, however, was also determined on spliced specimens at locations across the coupler, across the rebar-coupler interface, and on the abutting rebar. In most cases these measurements did not take place across a fracture. The initial gauge lengths on the coupler and across the rebar-coupler also varied in proportion to the coupler size. Initial gauge lengths placed on the abutting reinforcing steel, however, were always held constant at 8 inches.

As previously mentioned, slippage was determined by cycling specimens in tension to 50 percent of the specified yield, relaxing to 3,000 lb/in\(^2\), cycling again to 90 percent of the specified yield, and relaxing to 3,000 lb/in\(^2\). The deformation at 3,000 lb/in\(^2\) was considered
indicative of coupler slippage. Test results were interpreted using two different codes. For reinforcing bars No. 14 and smaller, the Standard Specifications for the State of California Department of Transportation require a total slip less than 0.010 inch after tensile loading to 50 percent of the specified yield of the unspliced reinforcing bar and relaxing to 3,000 lb/in². The Standard Specifications for Road and Bridge Construction for the State of Washington Department of Transportation require a total slip less than 0.045 inch after tensile loading to 90 percent of the specified yield of the unspliced reinforcing bar and relaxing to 3,000 lb/in².

DATA ANALYSIS

The primary objective of this testing program was to evaluate the performance of various mechanical splices against the basic and some of the more stringent requirements currently specified for mechanical connections. Nine different proprietary techniques from seven different manufacturers were tested. Tables 1 and 3 indicate that all grade 60, No. 9 spliced specimens, representing most proprietary splicing techniques, achieved the basic tensile strength requirement of 125 percent of the specified yield (75,000 lb/in²) as outlined by ACI 318-89 (section 12.14.3.4) and the 1984 AASHTO Standard Specifications for Highway Bridges.

In addition to the basic strength criterion, mechanical connections were also tested for conformance to more stringent strength and strain requirements outlined by others.

Table 5 contains data comparisons helpful in determining conformance of the spliced specimens to such requirements. The table does not include ultimate strength results from the third test series because the specimens were subjected to cyclic loading. Strain criterion comparisons shown in table 5 indicate that all spliced specimens met the criterion outlined by the State of California Department of Transportation by achieving ultimate tensile strengths in excess of 80,000 lb/in². In addition, all spliced specimens met the requirement specified by the United States Corps of Engineers and the State of Washington Department of Transportation by achieving tensile strengths greater than 0.9f_y (81,000 lb/in²).

The ACI 349-85 code, however, contains three different design criteria for mechanical splices including: basic strength or stress, strain (for staggering requirements), and cyclic. As previously mentioned, all specimens met the basic strength criterion. The cyclic criteria was beyond the scope of this research. Strain criterion comparisons of table 5 indicate the need for staggering spliced specimens from the DLN and LCT series if the computed design stress exceeds 0.5f_y (30,000 lb/in² for grade 60 rebar).

The ACI 359-86 code specifies the most stringent strength requirements: the tensile strength of the splice must not be less than 90 percent of the actual tensile strength of the reinforcing bar being tested, nor less than 100 percent of the specified minimum tensile strength (90,000 lb/in² for grade 60 rebar). Table 5 contains a column summarizing the result of 90 percent of the control bar strengths of test series four. Specimens WFC-2, DLNC-1, DBGTC-1, LCT-1, and LCC-1 did not achieve the required tensile strength of 90 percent of the control bar. In addition, specimens WFC-2, FH-1, and LCC-1 did not achieve the required strength of 90,000 lb/in².

With one exception, the connection types unable to meet this criterion typically used parallel threads or molten metal filler as the connecting mechanism. The FH specimens used tapered threads and did meet the strength requirement of 90 percent of the control bar strength, but
did not achieve the minimum required strength of 90,000 lb/in$^2$. The ultimate strength of the FH control bar was much lower than that of the other control bars. If the FH specimens had reinforcing bar strengths approaching 100,000 lb/in$^2$ rather than 90,000 lb/in$^2$, test results indicate they would have also likely passed. This likelihood is best exemplified in table 5 where the connection strength is expressed as a percentage of the control bar strength. In the case of the aforementioned conditions, the FH specimen, which achieved a tensile strength of 95.1 percent of the corresponding control bar, is one of the better performers.

Techniques that did not reduce the cross-sectional area of the rebar, such as cold-swaging, extrusion, hot-forging, tapered threads, or enlarged threads, did meet the criterion. None of the connections achieved an ultimate strength equivalent to their corresponding control bar strength. Strengths ranged from 80.9 percent to 97.6 percent when expressed in this manner.

Strength testing alone cannot always reveal the potential limitations of splicing systems, so another objective of the testing program was to obtain information useful in evaluating these limitations. To determine whether mechanical connections reduced the effective longitudinal stiffness of reinforcing steel in uniaxial tension, the first test series analyzed design assumptions by comparing reinforcing steel properties with connecting splice properties under the same loading conditions.

Examination of the curves and table 1 shows that in most cases, yielding of the reinforcing steel occurred simultaneously with that of the coupler, indicating that the connection behaved as a continuous unspliced bar. In a few cases, however, yielding occurred across the coupler first and was followed by yielding of the reinforcing steel (DLNC-1, LCT-1, LCC-1). In one of these cases (DFNC-1), the failure occurred at the coupler. In the other two cases, the failure occurred in the reinforcing bars. The specimen that failed in the coupler, however, still achieved a specified strength of 75,000 lb/in$^2$. Even though specified strengths were obtained, these results indicate that the splice reduced longitudinal stiffness in cases where the coupler yielded prior to the reinforcing bars tested in uniaxial tension.

The second and third test series were conducted to determine the slippage potential of the reinforcing steel within the coupler. Tables 2 and 3 indicate that the total slip exceeded the maximum permitted value of 0.010 inch in three cases where specimens were loaded to 50 percent of the specified yield and relaxed to 3,000 lb/in$^2$ (specimens DLNC-2, DLNC-3, WFC-4). Tables 2 and 3 also indicate that the total slip did not exceed the maximum permitted value of 0.045 inch in any case when specimens were loaded to 90 percent of the specified yield and relaxed to 3,000 lb/in$^2$.

The third test series also evaluated the longitudinal ductility (elongation) of the spliced specimens, which is critical to the seismic performance of any reinforced concrete structure. Table 3 indicates that the elongation across the coupler and the rebar-coupler interface was generally smaller than the elongation across the abutted rebar. However, elongation measurements did not take place across a fracture; all failures initiated outside the gauge lengths. Also, the initial gauge lengths across the coupler and the rebar-coupler interface varied in proportion to the size of the splice. Elongation comparisons over similar gauge lengths were therefore not obtained. The data comparisons of table 3 reveal that elongations measured across the reinforcing bars are generally greater than elongations measured across the rebar-coupler interface or across the coupler alone. Elongation measured across the reinforcing steel was usually double, and in a few cases five times greater than, the elongation measured.
across the rebar-coupler interface. Elongation across the coupler itself ranged from 0 to 3.3 percent. These values suggest that the elongation of spliced specimens restrained by the coupler will be less than that of a continuous unspliced bar in most cases.

The fourth test series was performed on nonspliced specimens to verify conformance to the requirements of ASTM A 615 for deformed bars. As table 4 indicates, tested specimens met the criteria of a minimum tensile strength of 90,000 lb/in$^2$, a minimum yield of 60,000 lb/in$^2$, and a minimum elongation of 7 percent over an 8-inch gauge length.

In some cases the failure of the spliced specimens occurred near an area where the LVDT plates were tack-welded to the rebar or where the surface was prepared for strain gage application. This phenomenon did not occur during the first test series because the plates were tack-welded onto the coupler ends rather than directly attached to the reinforcing steel. The failure of the unspliced specimen also sometimes occurred at a surface preparation area. Welding can have detrimental effects on reinforcing steel properties, but specimens were carefully prepared using minimal welding. Table 3 notes cases in which failure occurred at welded locations. The high strengths achieved by most spliced specimens indicate that tack welding and surface preparation used for this research had minimal effect on the rebar properties in most cases.
### Table 1.—First test series – reinforcing bar and connecting splice properties.

<table>
<thead>
<tr>
<th>Manufacturer strength design (lb/in²)</th>
<th>Specimen identification</th>
<th>Manufacturer</th>
<th>Yield at midpoint of coupler (lb)*</th>
<th>Yield across the coupler (lb)*</th>
<th>Yield of reinforcing steel (lb)*</th>
<th>Chord*** modulus of reinforcing steel (x10⁶ lb/in²)</th>
<th>Ultimate tensile strength (lb/in²)*</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>125 $f_y$ (75,000)</td>
<td>WFC-2</td>
<td>1</td>
<td>3-1/2</td>
<td>–</td>
<td>–</td>
<td>65,800</td>
<td>29.42</td>
<td>88,100</td>
</tr>
<tr>
<td>125 $f_y$ (75,000)</td>
<td>DLNC-1</td>
<td>2</td>
<td>5-3/4</td>
<td>**80,800</td>
<td>165,300</td>
<td>66,000</td>
<td>27.89</td>
<td>93,900</td>
</tr>
<tr>
<td>125 $f_y$ (75,000)</td>
<td>DEC-1</td>
<td>2</td>
<td>6-5/8</td>
<td>–</td>
<td>73,400</td>
<td>73,400</td>
<td>28.28</td>
<td>108,100</td>
</tr>
<tr>
<td>100 $f_i$ (90,000)</td>
<td>DBGC-1</td>
<td>3</td>
<td>6-1/2</td>
<td>–</td>
<td>–</td>
<td>69,400</td>
<td>27.34</td>
<td>107,200</td>
</tr>
<tr>
<td>125 $f_y$ (75,000)</td>
<td>DBGTC-1</td>
<td>3</td>
<td>8-3/4</td>
<td>**83,600</td>
<td>–</td>
<td>71,700</td>
<td>29.24</td>
<td>97,500</td>
</tr>
<tr>
<td>100 $f_i$ (90,000)</td>
<td>SC-1</td>
<td>4</td>
<td>5-1/2</td>
<td>**75,200</td>
<td>64,000</td>
<td>64,000</td>
<td>29.83</td>
<td>98,900</td>
</tr>
<tr>
<td>100 $f_i$ (90,000)</td>
<td>FHC-1</td>
<td>5</td>
<td>3-1/4</td>
<td>**76,800</td>
<td>63,900</td>
<td>63,900</td>
<td>26.94</td>
<td>86,900</td>
</tr>
<tr>
<td>150 $f_y$ (90,000)</td>
<td>RC-1</td>
<td>6</td>
<td>2-3/4</td>
<td>**75,300</td>
<td>–</td>
<td>71,600</td>
<td>29.44</td>
<td>100,600</td>
</tr>
<tr>
<td>100 $f_i$ (90,000)</td>
<td>LCT-1</td>
<td>7</td>
<td>5</td>
<td>–</td>
<td>**68,000</td>
<td>70,000</td>
<td>26.71</td>
<td>94,500</td>
</tr>
<tr>
<td>125 $f_y$ (75,000)</td>
<td>LCC-1</td>
<td>7</td>
<td>4</td>
<td>–</td>
<td>–</td>
<td>67,700</td>
<td>26.95</td>
<td>87,200</td>
</tr>
<tr>
<td>125 $f_y$ (75,000)</td>
<td>LC-1</td>
<td>7</td>
<td>3-3/4</td>
<td>**69,400</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>104,200</td>
</tr>
</tbody>
</table>

**NOTE:** All specimens loaded at 20,000 lb/in²/min.

*Data rounded to nearest 100 lb or 100 lb/in².

**Extension under load at 0.005 in/in.

***Upper and lower chord points set at 35,000 and 5,000 lb/in², respectively.

†Extension under load at 0.006 in/in.

‡Although 0.005 in/in was not obtained prior to LVDT removal, the curve indicates yield likely occurred simultaneously with the bar.
Table 2.—Second test series – slippage of reinforcing steel within the coupler.

<table>
<thead>
<tr>
<th>Specimen identification</th>
<th>Company</th>
<th>LVDT gauge length (in)</th>
<th>Slip at* 3,000 lb/in² after loading to 30,000 lb/in² (in)</th>
<th>Slip at** 3,000 lb/in² after loading to 54,000 lb/in² (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>WFC-3</td>
<td>1</td>
<td>7-1/2</td>
<td>0.0047</td>
<td>0.0089</td>
</tr>
<tr>
<td>DLNC-2</td>
<td>2</td>
<td>10</td>
<td>0.0106</td>
<td>0.0185</td>
</tr>
<tr>
<td>DEC-2</td>
<td>2</td>
<td>10-3/4</td>
<td>0.0012</td>
<td>0.0034</td>
</tr>
<tr>
<td>DBG-2</td>
<td>3</td>
<td>10-1/2</td>
<td>0.0011</td>
<td>0.0012</td>
</tr>
<tr>
<td>DBGTC-2</td>
<td>3</td>
<td>13</td>
<td>0.0031</td>
<td>0.0070</td>
</tr>
<tr>
<td>SC-2</td>
<td>4</td>
<td>9-3/4</td>
<td>0.0016</td>
<td>0.0116</td>
</tr>
<tr>
<td>FHC-2</td>
<td>5</td>
<td>7-1/2</td>
<td>0.0012</td>
<td>0.0024</td>
</tr>
<tr>
<td>RC-2</td>
<td>6</td>
<td>7</td>
<td>0.0035</td>
<td>0.0063</td>
</tr>
<tr>
<td>LCT-2</td>
<td>7</td>
<td>9-3/16</td>
<td>0.0064</td>
<td>0.0268</td>
</tr>
<tr>
<td>LCC-2</td>
<td>7</td>
<td>8-5/16</td>
<td>0.0035</td>
<td>0.0399</td>
</tr>
<tr>
<td>LT-2</td>
<td>7</td>
<td>7-3/4</td>
<td>0.0017</td>
<td>0.0026</td>
</tr>
</tbody>
</table>

NOTE: All specimens loaded at 20,000 lb/in²/min.

*Slip after loading to 50 percent of minimum specified yield. Maximum allowable slip = 0.010 in.

**Slip after loading to 90 percent of minimum specified yield. Maximum allowable slip = 0.045 in.
<table>
<thead>
<tr>
<th>Manufacturer strength design (lb/in²)</th>
<th>Specimen identification</th>
<th>Specimen identification</th>
<th>Company</th>
<th>gauge length (in)</th>
<th>Slip at* LVDT after loading to 30,000 lb/in² (in)</th>
<th>Elongation across rebar-coupler interface (%)</th>
<th>Elongation across rebar (%)</th>
<th>Ultimate tensile strength (lb/in²)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>125 fᵧ (75,000)</td>
<td>WFC-4</td>
<td>1</td>
<td>7-11/16</td>
<td>0.0162</td>
<td>0.0202</td>
<td>3.1</td>
<td>4.6</td>
<td>88,100</td>
<td>Bar threads</td>
</tr>
<tr>
<td>125 fᵧ (75,000)</td>
<td>DLNC-3</td>
<td>2</td>
<td>10</td>
<td>0.0230</td>
<td>0.0332</td>
<td>0</td>
<td>1.6</td>
<td>***84,500</td>
<td>Bar</td>
</tr>
<tr>
<td>125 fᵧ (75,000)</td>
<td>DEC-3</td>
<td>2</td>
<td>10-15/16</td>
<td>0.0074</td>
<td>0.0098</td>
<td>0</td>
<td>0.8</td>
<td>***83,000</td>
<td>Bar</td>
</tr>
<tr>
<td>100 ft (90,000)</td>
<td>DBGC-3</td>
<td>3</td>
<td>10-9/16</td>
<td>0.0023</td>
<td>0.0059</td>
<td>0</td>
<td>1.5</td>
<td>***107,200</td>
<td>Bar</td>
</tr>
<tr>
<td>125 fᵧ (75,000)</td>
<td>DBGTC-3</td>
<td>3</td>
<td>13</td>
<td>0.0067</td>
<td>0.0105</td>
<td>3.1</td>
<td>0</td>
<td>***76,400</td>
<td>Bar</td>
</tr>
<tr>
<td>100 ft (90,000)</td>
<td>SC-3</td>
<td>4</td>
<td>9-11/16</td>
<td>0.0043</td>
<td>0.0097</td>
<td>0</td>
<td>3.4</td>
<td>***89,000</td>
<td>Bar</td>
</tr>
<tr>
<td>100 ft (90,000)</td>
<td>FHC-3</td>
<td>5</td>
<td>7-7/16</td>
<td>0.0011</td>
<td>0.0034</td>
<td>3.1</td>
<td>3.1</td>
<td>82,700</td>
<td>Bar threads</td>
</tr>
<tr>
<td>150 fᵧ (90,000)</td>
<td>RC-3</td>
<td>6</td>
<td>6-13/16</td>
<td>0.0072</td>
<td>0.0092</td>
<td>3.3</td>
<td>2.6</td>
<td>99,600</td>
<td>Bar</td>
</tr>
<tr>
<td>100 ft (90,000)</td>
<td>LCT-3</td>
<td>7</td>
<td>9-1/8</td>
<td>0.0033</td>
<td>0.0344</td>
<td>3.1</td>
<td>6.3</td>
<td>102,300</td>
<td>Bar</td>
</tr>
<tr>
<td>125 fᵧ (75,000)</td>
<td>LCC-3</td>
<td>7</td>
<td>8-1/16</td>
<td>0.0088</td>
<td>0.0335</td>
<td>0</td>
<td>4.1</td>
<td>97,100</td>
<td>Bar</td>
</tr>
<tr>
<td>125 fᵧ (75,000)</td>
<td>LC-3</td>
<td>7</td>
<td>7-11/16</td>
<td>0.0018</td>
<td>0.0029</td>
<td>0</td>
<td>2.2</td>
<td>***102,300</td>
<td>Bar</td>
</tr>
</tbody>
</table>

NOTE: Elongation measurements were not across a fracture. All specimens loaded at 20,000 lb/in²/min.

*Slip after loading to 50 percent of minimum specified yield. Maximum allowable slip = 0.010 in.
**Slip after loading to 90 percent of minimum specified yield. Maximum allowable slip = 0.045 in.
***Specimen failed where plate was tack-welded to reinforcing steel.
†Elongation was measured across a fracture.
Table 4.—Fourth test series – reinforcing bar properties.

<table>
<thead>
<tr>
<th>Specimen identification</th>
<th>Elongation across rebar (%)</th>
<th>Yield* rebar (lb/in²)</th>
<th>Ultimate* strength (lb/in²)</th>
<th>Chord** modulus of elasticity (×10⁶ lb/in²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>WF1</td>
<td>15.6</td>
<td>66,900</td>
<td>98,800</td>
<td>28.59</td>
</tr>
<tr>
<td>DLN1</td>
<td>17.2</td>
<td>66,900</td>
<td>108,000</td>
<td>28.54</td>
</tr>
<tr>
<td>DE1</td>
<td>10.2</td>
<td>77,300</td>
<td>117,500</td>
<td>28.42</td>
</tr>
<tr>
<td>DBG1</td>
<td>10.6</td>
<td>75,400</td>
<td>112,400</td>
<td>27.83</td>
</tr>
<tr>
<td>DBGT1</td>
<td>9.8</td>
<td>67,600</td>
<td>111,600</td>
<td>29.62</td>
</tr>
<tr>
<td>S1</td>
<td>10.6</td>
<td>65,700</td>
<td>103,600</td>
<td>28.19</td>
</tr>
<tr>
<td>FH1</td>
<td>13.3</td>
<td>64,500</td>
<td>90,200</td>
<td>26.56</td>
</tr>
<tr>
<td>R1</td>
<td>13.7</td>
<td>72,700</td>
<td>103,100</td>
<td>28.97</td>
</tr>
<tr>
<td>L1</td>
<td>9.4</td>
<td>73,100</td>
<td>107,800</td>
<td>27.52</td>
</tr>
<tr>
<td>Requirements</td>
<td>7.0 min</td>
<td>60,000 min</td>
<td>90,000 min</td>
<td>–</td>
</tr>
</tbody>
</table>

*Data rounded to nearest 100 lb/in².

**Upper and lower chord points set at 35,000 and 5,000 lb/in², respectively.
Table 5.—Mechanical connection conformance to codes.

<table>
<thead>
<tr>
<th>Specimen identification</th>
<th>Ultimate strength of spliced specimen (lb/in²)</th>
<th>Ultimate strength of reinforcing bar (lb/in²)</th>
<th>Spliced specimen strength as a percentage of reinforcing bar strength (%)</th>
<th>90% of reinforcing bar strength (lb/in²)</th>
<th>Strain across coupler at 0.9 fy (×10⁻⁶ in/in)</th>
<th>1.5 times the reinforcing bar strain at 0.9 fy (×10⁻⁶ in/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>WF series</td>
<td>88,100</td>
<td>98,800</td>
<td>89.2</td>
<td>88,920</td>
<td>1,530</td>
<td>2,870</td>
</tr>
<tr>
<td>DLN series</td>
<td>93,900</td>
<td>108,000</td>
<td>86.9</td>
<td>97,200</td>
<td>3,910</td>
<td>2,860</td>
</tr>
<tr>
<td>DE series</td>
<td>105,100</td>
<td>117,500</td>
<td>92.0</td>
<td>105,750</td>
<td>990</td>
<td>2,860</td>
</tr>
<tr>
<td>DBG series</td>
<td>107,200</td>
<td>112,400</td>
<td>98.4</td>
<td>101,160</td>
<td>960</td>
<td>2,930</td>
</tr>
<tr>
<td>DBGT series</td>
<td>97,500</td>
<td>111,600</td>
<td>87.4</td>
<td>100,440</td>
<td>1,300</td>
<td>2,740</td>
</tr>
<tr>
<td>S series</td>
<td>98,900</td>
<td>103,600</td>
<td>95.1</td>
<td>93,240</td>
<td>1,010</td>
<td>2,930</td>
</tr>
<tr>
<td>FH series</td>
<td>85,800</td>
<td>90,200</td>
<td>96.1</td>
<td>81,180</td>
<td>1,670</td>
<td>3,060</td>
</tr>
<tr>
<td>R series</td>
<td>100,600</td>
<td>103,100</td>
<td>97.6</td>
<td>92,790</td>
<td>410</td>
<td>2,090</td>
</tr>
<tr>
<td>LT series</td>
<td>94,500</td>
<td>107,800</td>
<td>87.7</td>
<td>97,020</td>
<td>2,710</td>
<td>2,920</td>
</tr>
<tr>
<td>LC series</td>
<td>87,200</td>
<td>107,800</td>
<td>80.9</td>
<td>97,020</td>
<td>3,600</td>
<td>2,920</td>
</tr>
<tr>
<td>L series</td>
<td>104,200</td>
<td>107,800</td>
<td>96.7</td>
<td>97,020</td>
<td>1,780</td>
<td>2,920</td>
</tr>
</tbody>
</table>

Codes and criteria:


4. ACI 359-86 Code for Concrete Reactor Vessels and Containments:

   (1) Strength of spliced specimen ≥ 0.90 (strength of control reinforcing bars).

   (2) Strength of spliced specimen ≥ 1.00 f₁ (90,000 lb/in²).

5. ACI 349-85 Code Requirements for Nuclear Safety Related Concrete Structures:

   (1) Strength of spliced specimen ≥ 1.25 fy (75,000 lb/in²).

   (2) If computed design stress ≥ 0.5 fy (30,000 lb/in²), then splices staggered if strain across coupler at 0.9 fy (54,000 lb/in²) ≥ 1.5 (strain of control bars) at 0.9 fy (54,000 lb/in²).

   (3) Conformance to cyclic criteria not tested.
Figure 1
USBR CONCRETE LABORATORY
MECHANICAL SPICING TESTING
07/27/89

TEST COMPANY #1
SPECIMEN WFC-2
COUPLER LVDT'S
COUPLER GAGES
BAR GAGES

LOAD (POUNDS)

0  500  1000  1500  2000  2500  3000  3500  4000  4500  5000
STRAIN (microstrain)

Figure 2
USBR CONCRETE LABORATORY
MECHANICAL SPICING TESTING
07/27/89

TEST: COMPANY#2
SPECIMENT: DLNC-1
COUPLER LVDT'S
COUPLER GAGES
BAR GAGES

Figure 3
Figure 5

USBR CONCRETE LABORATORY
MECHANICAL SPLICING TESTING
10/19/89

TEST COMPANY #2
SPECIMEN DEC-1
COUPLER LVDT'S
COUPLER GAGES
BAR GAGES

LOAD (POUNDS)

0 5000 10000 15000 20000 25000 30000 35000 40000 45000 50000

STRAIN (microstrain)
Figure 6
Figure 7
Figure 8
Figure 9

LOAD (POUNDS)

0  5000  10000  15000  20000  25000  30000  35000

STRAIN (microstrain)

0  10000  20000  30000  40000  50000  60000  70000  80000  90000  100000  110000

COUPLER LVDT'S
COUPLER GAGES
BAR GAGES

TEST COMPANY #3
SPECIMEN: DBGTC-1
07/28/89
USBR CONCRETE LABORATORY
MECHANICAL SPICING TESTING
07/28/89

Figure 10
USBR CONCRETE LABORATORY
MECHANICAL SPLICING TESTING
07/31/89

TEST COMPANY #4
SPECIMEN SC-1

COUPLER LVDT'S
COUPLER GAGES
BAR GAGES

LOAD (POUNDS)
0 10000 20000 30000 40000 50000 60000

STRAIN (microstrain)
0 5000 10000 15000 20000 25000 30000 35000 40000 45000 50000

Figure 11
USBR CONCRETE LABORATORY
MECHANICAL SPlicing TESTING
07/31/89

Figure 12
Figure 13
USBR CONCRETE LABORATORY
MECHANICAL SPLICING TESTING
07/31/89

TEST COMPANY#5
SPECIMEN FHC-1
COUPLER LVDT'S
COUPLER GAGES
BAR GAGES

LOAD (POUNDS)

80000
70000
60000
50000
40000
30000
20000
10000
0

0 500 1000 1500 2000 2500 3000 3500 4000 4500 5000

STRAIN (microstrain)

Figure 14
Figure 15
USBR CONCRETE LABORATORY
MECHANICAL SPLICING TESTING
08/02/89

TEST COMPANY #6
SPECIMEN RC-1

STRAIN (microstrain)

LOAD (POUNDS)

0 500 1000 1500 2000 2500 3000 3500 4000 4500 5000

0 10000 20000 30000 40000 50000 60000 70000 80000 90000 100000

Figure 16
Figure 20
Figure 22
USBR CONCRETE LABORATORY
MECHANICAL SPICING TESTING
SPECIMEN: WFC-3
9/20/89

Figure 23
USBR CONCRETE LABORATORY
MECHANICAL SPlicing TESTING
9/20/89

TEST COMPANY#2
SPECIMEN: DLNC-2

LOAD (POUNDS)

DEFORMATION (INCHES)

Figure 24
Figure 25
USBR CONCRETE LABORATORY
MECHANICAL SPlicing TESTING
9/19/89

Figure 26

DEFORMATION (INCHES)
USBR CONCRETE LABORATORY
MECHANICAL SPICING TESTING
9/20/89

TEST COMPANY#4
SPECIMEN: SC-2

Figure 28

DEFORMATION (INCHES)
Figure 29
Figure 30
Figure 32
Figure 33
Figure 34
USB.R CONCRETE LABORATORY
MECHANICAL SPICING TESTING
08/23/91

TEST: COMPANY #2
SPECIMEN: DLNC-3

Figure 35
USBR CONCRETE LABORATORY
MECHANICAL SPICING TESTING
08/23/91

TEST: COMPANY#3
SPECIMEN: DBGC-3

DEFORMATION (INCHES)

LOAD (POUNDS)

Figure 37
Figure 38
USBR CONCRETE LABORATORY
MECHANICAL SPlicing TESTING
08/22/91

TEST: COMPANY#4
SPECIMEN: SC-3

LOAD (POUNDS)
0 10000 20000 30000 40000 50000 60000 70000 80000 90000

DEFORMATION (INCHES)
0.000 0.010 0.020 0.030 0.040 0.050 0.060 0.070 0.080 0.090 0.100

Figure 39
Figure 43
DATE: 01/23/90
USBR CONCRETE LABORATORY

TEST: REBAR PROPERTIES
SPECIMEN: WF-1
ULTIMATE STRENGTH: 98800 LB/IN^2

Figure 45
DATE: 01/23/90
USB CONCRETE LABORATORY

TEST: REBAR PROPERTIES
SPECIMEN: DLN-1
ULTIMATE STRENGTH: 108000 LB/IN^2

STRESS (LB/IN^2)

0 10000 20000 30000 40000 50000 60000 70000 80000 90000 100000 110000

0 5000 10000 15000 20000 25000 30000 35000 40000 45000 50000

STRAIN (microstrain)

Figure 46
DATE: 01/23/90
USBR CONCRETE LABORATORY
TEST: REBAR PROPERTIES
SPECIMEN: DE-1
ULTIMATE STRENGTH: 117500 LB/IN²

STRESS (LB/IN²)

0 10000 20000 30000 40000 50000 60000

0 5000 10000 15000 20000 25000 30000 35000 40000 45000 50000 55000 60000

STRAIN (microstrain)

Figure 47
DATE: 01/23/90
USBR CONCRETE LABORATORY

TEST: REBAR PROPERTIES
SPECIMEN: DBG-1
ULTIMATE STRENGTH: 112400 LB/IN^2

Figure 48
DATE: 01/23/90
USBR CONCRETE LABORATORY

TEST: REBAR PROPERTIES
SPECIMEN: DBGT-1
ULTIMATE STRENGTH: 111600 LB/IN²

Figure 49
DATe: 01/23/90
USBR CONCRETE LABORATORY

TEST: REBAR PROPERTIES
SPECIMEN: S-1
ULTIMATE STRENGTH: 103600 LB/IN^2

STRESS (LB/IN^2)

0 10000 20000 30000 40000 50000 60000 70000 80000
0 10000 20000 30000 40000 50000 60000 70000 80000

STRAIN (microstrain)

Figure 50
DATE: 01/23/90
TEST: REBAR PROPERTIES
USBR CONCRETE LABORATORY
SPECIMEN: FH-1
ULTIMATE STRENGTH: 90200 LB/IN^2

STRESS (LB/IN^2)

STRAIN (microstrain)

Figure 51
DATE: 01/23/90
USBR CONCRETE LABORATORY

TEST: REBAR PROPERTIES
SPECIMEN: R-1
ULTIMATE STRENGTH: 103100 LB/IN²

Figure 52
DATE: 01/23/90
USBR CONCRETE LABORATORY
TEST: REBAR PROPERTIES
SPECIMEN: L-1
ULTIMATE STRENGTH: 107800 LB/IN²

Figure 53
APPENDIX

Description of proprietary tension splicing systems

Only the engineer familiar with overall structural design, probable construction conditions, final conditions of service, and the unique requirements of proprietary mechanical splicing techniques can properly evaluate the variables to select the most efficient and economical splicing method. To help the engineer make a selection and interpret the test results contained in this report, descriptions of the various proprietary mechanical splicing techniques are in order. The descriptions will outline the splice method, the preparation of bar ends, the spacing and clearance considerations, and any other special considerations unique to the individual splicing operation. Many of these tension splicing systems also satisfy compression splice requirements.

Proprietary mechanical splicing techniques may require specialized installation equipment or procedures that can influence design and construction methods. Many couplers have an outside diameter substantially larger than the spliced reinforcing bars. The size and operation of the equipment required to make mechanical connections can therefore dictate a minimum spacing or stagger pattern. Selecting and positioning mechanical connections demands consideration of clearance limits between spliced rebar and required minimum concrete cover over stirrups, ties, spirals, and dowels at spliced locations. The selection of appropriate mechanical splicing methods may also be constrained by the following construction considerations:

- field erection of freestanding or preassembled cages
- preparation of bar ends prior to splicing
- potential damage during splicing of coated reinforcing steel
- accessibility and availability of materials and skilled labor
- seismic design factors

ACI 439.3R contains a synopsis the reader should consult for more detailed information regarding a particular splicing system. This document incorporates the terminology used in ACI 439.3R to provide consistency in describing various mechanical splicing devices.

Figure A.1. – Cold-swaged steel coupling sleeve (company 3) – DBGC specimens

A cold-swaged steel sleeve splice involves slipping a seamless steel tube over the abutting ends of two reinforcing bars and deforming the tube to the bar configuration. A coupler may be applied onsite or in the fabrication plant using portable presses ranging in size and weight (20 to 230 pounds) in proportion to the diameter of the reinforcing steel. A coupler may also be applied to one of the bar ends prior to shipment to the jobsite using a nonportable bench press. The splicing operation involves inserting one reinforcing bar halfway into the sleeve. Using a two-piece die set, the sleeve is hydraulically pressed onto the bar in lengthwise segments with a series of overlapping pressings. The second reinforcing bar is then inserted into the sleeve at the jobsite and the die set is again used to hydraulically press the sleeve onto the second bar with overlapping pressings, thereby completing the splice. Splices may be performed on reinforcing steel ranging in size from No. 3 to 18. Transition splices are available to connect different bar sizes. The splicing sleeves vary in diameter and length in proportion to the size of the spliced rebar.
The bars do not require special end preparation. The ends may be sheared or flame cut. However, burrs or other surface imperfections that could interfere with sleeve placement must be removed.

In making a mechanical connection, consideration must be given to the size and operation of equipment required as well as the outside diameter of a splice. These factors can dictate the minimum spacing or stagger pattern of the reinforcing steel. The diameter of the coupling system is large enough to require adjustment of stirrup or tie spacings adjacent to the splice, thereby providing the required concrete cover over the bars.

Figure A.2. — Extruded steel coupling sleeve (company 2) — DEC specimens

An extrusion press is used at the construction site to make this splice. The size and weight of the extrusion press (125 to 1,000 lb) varies in proportion to the size of the spliced reinforcing steel. The splicing operation involves centering the coupler sleeve over the butted ends of the reinforcing steel and fixing the coupler to one rebar with a set screw. The hydraulic or extrusion press then pushes a drawing die over the entire length of the coupler in one operation. Splices may be performed on reinforcing bars ranging in size from No. 5 to 18. Transition couplers are available for splicing bars of the next lower or larger size. The splicing sleeves vary in diameter and length in proportion to the size of the spliced rebar.

The bars do not require special end preparation. The ends may be sheared or flame cut. However, burrs or other surface imperfections that could interfere with sleeve placement must be removed.

In making a mechanical connection, consideration must be given to the size and operation of equipment required as well as the outside diameter of a splice. These factors can dictate the minimum spacing or stagger pattern of the reinforcing steel. The diameter of the coupling system is large enough to require adjustment of stirrup or tie spacings adjacent to the splice, thereby providing the required concrete cover over the bars.

Figure A.3. — Cold-swaged steel coupling sleeves with threaded ends acting as a coupler (company 3) — DBGTC specimens

Splicing with a cold-swaged steel sleeve with threaded ends involves cold-swaging the nonthreaded ends of two separate sleeves onto the ends of two reinforcing bars prior to field assembly of the entire structure. An internally threaded female coupler is cold forged onto the end of one rebar while a prethreaded male adapter, matching the internal threads of the female coupler, is swaged onto the second rebar. The bars are aligned and twisted together onsite to complete the splice. The net area through the thread region equals or exceeds the nominal bar diameter. Torquing is generally not specified. Reinforcing bars with different diameters may also be connected in this manner.

Another variation on this splice uses two internally threaded couplers swaged onto the end of each rebar. Couplers are interconnected with a prethreaded high-strength steel stud turned to draw the reinforcing steel together using a turnbuckle effect. The connection is useful when assembly prohibits turning the rebar.

Splicing bars using these systems requires three connections to splice two rebars together. Sleeves are available to splice rebar ranging in size from No. 3 to 18. Transitions from one bar
size to another or connections to structural steel can also be made. The sleeves vary in diameter and length in proportion to the size of the spliced rebars.

The bars do not require special end preparation. The ends may be sheared or flame cut. However, burrs or other surface imperfections that could interfere with sleeve placement must be removed.

The diameter of the sleeve is large enough to require adjustment of stirrup or tie spacings adjacent to the splice, thereby providing the required concrete cover over the bars.

Notch effects caused by bar threading must be considered if the connection is susceptible to seismic excitation, dynamic or fatigue loadings, or cold temperatures.

Figure A.4. – Hot-forged steel coupling sleeve (company 4) – SC specimens

A hot-forged steel sleeve splice involves placing a preheated steel sleeve over the abutting ends of two reinforcing bars and deforming the sleeve to the bar configuration. Sleeves are heated in a gas furnace to approximately 2,000 °F. A support clamping fixture sets the hydraulic press in position on the reinforcing steel, and the heated steel sleeve is placed halfway over one of the rebars. The opposing rebar is then placed into the splicing sleeve and the hydraulic press forges the sleeve into the deformations of both bars. Contraction of the sleeve upon cooling improves the bond and increases splice strength.

Sleeves are available to splice rebars ranging in size from No. 5 to 18. The sleeves vary in diameter and length in proportion to the diameter of the reinforcing bar being spliced. Transition couplers are available for connecting different rebar sizes.

The bars do not require special end preparation. The ends may be sheared or flame cut. However, burrs or other surface imperfections that could interfere with sleeve placement must be removed.

In making a mechanical connection, consideration must be given to the size and operation of equipment required as well as the outside diameter of a splice. These factors can dictate the minimum spacing or stagger pattern of the reinforcing steel. The diameter of the coupling system is large enough to require adjustment of stirrup or tie spacings adjacent to the splice, thereby providing the required concrete cover over the bars.

The hot-forged system requires a furnace and fuel source near the immediate work location, so the splice locations must be considered when using this method.

Figure A.5. – Coupler for thread-deformed reinforcing bars (company 2) – DLNC specimens

This splice uses coupler sleeves manufactured with internal threads that match the rolled threads of the reinforcing steel. The reinforcing threads are hot-rolled over the entire length of the rebar and conform to the requirements of ASTM A 615 except for a lack of markings on the reinforcing steel indicating the rebar size and the fabricating mill. In installations requiring rotation of one reinforcing bar, a splice is performed by threading the coupler onto one bar and torquing the opposing rebar into the coupler. In installations where neither bar may be turned, a coupler is engaged on the ends of two opposing rebars and a hex or lock-jam nut is torqued on
each end of the coupler with hydraulic wrenches. The largest rebar requires torque wrenches of up to 3,000 ft-lb in capacity. Hex nuts are specified in cases requiring tension and compression splice strengths of 125 percent of the minimum specified yield of the reinforcing steel. Lock nuts are specified in cases requiring tension splice strengths of 125 percent of the minimum specified yield and compression splice strengths of half this amount. Couplers and jam nuts vary in diameter and length in proportion to the size of rebar being spliced and are available for rebar ranging in size from No. 6 to 18.

The bars do not require special end preparation. The ends may be sheared, sawed, or flame cut. However, burrs, scale, or rust must be removed to permit proper engagement of the coupler and jam-nut threads on the bar deformations. Concerns involving rebar availability, identification, and traceability must be addressed prior to construction because the coupling system requires the use of specialized reinforcing steel without standardized markings.

The diameter of the coupling system is large enough to require adjustment of stirrup or tie spacings adjacent to the splice, thereby providing the required concrete cover over the bars.

Notch effects caused by bar threading must be considered if the connection is susceptible to seismic excitation, dynamic or fatigue loadings, or cold temperatures.

Figure A.6 - Taper-threaded steel coupler (companies 5 and 7) – LC and FHC specimens

Taper-threaded steel splices are achieved by threading a coupler with internal tapered threads onto one rebar and twisting the matching external threads of the opposing rebar into the coupler. To complete a splice using this method, the ends of the straight rebar must be free to rotate. Special couplers are available for connecting curved bars where the ends cannot be rotated. The increased thread area of the tapered thread provides a greater load capacity than a similar system containing parallel threads. The tapered thread will also minimize stress concentrations caused by threading, as well as potential cross-threading, jamming, or thread damage prior to full thread engagement. Full thread engagement generally requires torquing of the connection to 200 ft-lb.

Two manufacturers make this coupler. The threads used by each manufacturer differ enough to prevent product interchangeability. Both manufacturers also make couplers to connect reinforcing bars to other structural members and transition from one bar size to another. Couplers are available to splice reinforcing steel ranging in size from No. 4 to 18 from one manufacturer and from No. 7 to 18 from the other manufacturer. The couplers vary in diameter and length in proportion to the size of the spliced rebar.

Machine threading performed in the field or at the fabrication plant requires special end preparation of the rebar. The threaded ends must be protected from damage during shipping and handling.

The diameter of the coupler is large enough to require adjustment of stirrup and tie spacings adjacent to the splice, thereby providing the required concrete cover over the bars.

Notch effects caused by bar threading must be considered if the connection is susceptible to seismic excitation, dynamic or fatigue loadings, or cold temperatures.
Another splicing system incorporates a steel coupler with internal threads that match the machined or rolled parallel threads of the opposing reinforcing steel. A positive stop at the coupler center assures proper thread engagement. Splicing is achieved by threading the coupler onto one rebar and twisting the matching rebar into the coupler. The coupler is generally connected to one reinforcing bar prior to placement. A pipe wrench may be used to tighten the bar to the stops. When codes require a splice of full ultimate tensile strength equaling that of a nonthreaded rebar, a larger diameter rebar may be used to offset the reduction in the rebar cross-section caused by threading. Couplers vary in size and length in proportion to the spliced rebar and are available to splice rebars ranging in size from No. 4 to 18.

Machine threading, performed in the field or at the fabrication plant, requires special end preparation of the rebar. The threaded ends must be protected from damage during shipping and handling.

The diameter of the coupler is large enough to require adjustment of stirrup or tie spacings adjacent to the splice, thereby providing the required concrete cover over the bars.

Notch effects caused by bar threading must be considered if the connection is susceptible to seismic excitation, dynamic or fatigue loadings, or cold temperatures.

This system is used primarily to achieve a doweled splice at a construction joint, allowing reinforcement continuity without formwork penetration or reinforcing steel projection. Four different dowel bar connection systems currently exist. In all systems, the coupler is modified to include a flange with holes allowing attachment to formwork. The coupler is commonly threaded onto one rebar. After concrete placement and formwork removal, a second rebar may be threaded into the coupler to continue the reinforcement. Three systems are extensions of systems previously discussed, including the coupler with standard threads, the coupler for thread-deformed reinforcing bars, and the cold-swaged steel coupler.

The fourth system incorporates a coupler forged with an internally threaded barrel. The thread size is larger than the nominal size of the spliced rebar. The end of the adjoining piece or "dowel in" is enlarged by forging before external threading to prevent reduction of cross-sectional rebar area. The splicing system therefore develops the minimum specified reinforcing steel strength of at least 150 percent of the minimum specified yield. A splice is accomplished in one connection by hand-threading the two pieces together. This system requires no wrenches, torquing, or other special equipment, and can accommodate rebar sizes ranging from No. 4 to 11. Splice size varies in length and diameter in proportion to the size of the spliced rebar.

The system is forged from 100-percent reinforcing steel material and is prepackaged, so no bar end preparations are necessary in the field. Male threads, however, may be cut in the field to meet schedule changes or emergencies. Details involving bar and threading equipment availability must therefore be worked out prior to construction.
The diameter of the forged rebar is large enough to require adjustment of stirrup or tie spacings adjacent to the splice, thereby providing the required concrete cover over the bars.

Notch effects caused by bar threading must be considered if the connection is susceptible to seismic excitation, dynamic or fatigue loadings, or cold temperatures.

**Figure A.9. – Steel-filled coupling sleeve (company 7) – LCT and LCC specimens**

The steel-filled coupling splice involves centering two reinforcing bars end-to-end in an internally grooved steel tube and introducing a molten metal filler through a tap hole. As the molten material solidifies, a load transferring interlock forms between the grooves inside the sleeve and the deformations on the reinforcing steel. Sleeves and equipment are available to perform horizontal or vertical splices in compression only. Sleeves and equipment are also available to perform tension splices to 125 or 150 percent of the minimum specified yield. Splices can be conducted on rebar sizes ranging from No. 4 to 18. Sleeve sizes vary in diameter and length in proportion to the size of the spliced rebar. Transition splices for connecting different bar sizes are also available, as are splices for connections to structural steel.

The rebar does not require special end preparation, but the splice ends must be dry and clean. Burrs or other surface imperfections that interfere with sleeve placement must be removed. Bars may be shear cut, flame cut, or saw cut. Because inspection is limited to the exterior of the splice, proper installation is important.

In making a mechanical connection, consideration must be given to the size and operation of equipment required and to the outside diameter of a splice. These factors can dictate the minimum spacing or stagger pattern of the reinforcing steel. The sleeve diameter is large enough to require adjustment of adjacent stirrups or tie spacings to provide the required concrete cover over the bars.

Consideration must also be given to the environmental job-site requirements and fire protection required by the exothermic reaction of the splicing operation.

**Grout-filled coupling sleeve (company 8)**

The grout-filled coupling sleeve is customarily used to connect precast reinforced structural members and has had some application in cast-in-place concrete. To perform a splice, reinforcing bars are inserted into an internally grooved sleeve and butted end to end at the sleeve center. An expansive, nonshrink, high-strength proprietary cement grout is then introduced (by a low-pressure pump) to form a load transferring interlock between the reinforcing steel deformations and the internal grooves of the steel sleeve. Because the splice strength depends upon the embedment length and resulting bond between the reinforcing steel and the high-strength grout, the sleeve is much longer (up to 14 bar diameters) and larger in diameter (up to 4 inches) than other splices. Grout setting time may range from 2 to 4 hours or longer, depending on the ambient temperature.

Sleeves are available to perform splices for full positive tensile connections (125 percent of the specified minimum yield for grade 60 rebar) and for positive tensile connections used in regions of low computed stresses (100 percent of the specified minimum yield for grade 60 rebar). In compression, both splices also conform to code requirements for full positive connections. Splices may be performed on reinforcing steel ranging in size from No. 5 to 18 as well on
transitions or combinations of different rebar sizes. Sleeve sizes vary in diameter and length in proportion to the size of the spliced rebar.

The rebar does not require special end preparation, but care must be taken to avoid movement of the reinforcing steel prior to initial set and sufficient grout strength gain. Rebars may be saw cut, shear cut, or flame cut.

The diameter of the sleeves is large enough to require adjustment of stirrup or tie spacings adjacent to the splice, thereby providing the required concrete cover over the bars.

**Steel coupling sleeve with wedge (company 9)**

This new mechanical splice, introduced to the market shortly after procurement and testing of the specimens used in this research program, is mentioned here for completeness. This system is still experimental because of limited use in situations requiring tension reinforcement and is still under evaluation for use in compression. The splice is designed for mechanical connections between small diameter rebar ranging in size from No. 3 to 7. To perform a splice, two reinforcing bars are overlapped in an oval coupling sleeve such that the bars extend beyond the sleeve end approximately one bar diameter. A wedge-shaped pin is then driven through an insertion hole in the sleeve such that it passes between the reinforcing steel and out a hole opposite the insertion hole. A hydraulic device drives the pin.
Figure A.1. – Cold-swaged steel coupling sleeve – DBGC specimens (company 3).
Figure A.2. – Extruded steel coupling sleeve – DEC specimens (company 2).
Figure A.3. – Cold-swaged steel coupling sleeves with threaded ends acting as a coupler – DBGTC specimens (company 3).
Figure A.4. – Hot-forged steel coupling sleeve – SC specimens (company 4).
Figure A.5. – Coupler for thread-deformed reinforcing bars – DLNC specimens (company 2).
Figure A.6. - Taper-threaded steel coupler – FHC and LC specimens (companies 5 and 7, respectively).
Figure A.7. - Threaded couplers with standard national coarse threads – WFC specimens (company 1). The bar on the right has been positioned to expose the threads within the coupler.
Figure A.8. – Forged threaded steel coupler with standard rolled threads (dowel bar mechanical connection systems – RC specimens) (company 6).
Figure A.9  Steel-filled coupling sleeve – LCT and LCC specimens (company 7)
Mission

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