Bending/Straightening and Grouting Concrete Reinforcing Steel:

Review of Washington State Department of Transportation's Specifications and Proposed Modifications

WA-RD 168.1

Final Report
December 1988

Washington State Department of Transportation
Planning, Research and Public Transportation

in cooperation with the
United States Department of Transportation
Federal Highway Administration

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This report consists of two volumes. The first volume reviews the WSDOT's specification guidelines for field bending/straightening concrete reinforcing bars, and proposes modifications to those guidelines based on current knowledge. The second volume reviews the WSDOT's specification guidelines for resin grouting epoxy-coated concrete reinforcing bars, and suggests modifications to the guidelines for grouting based on the available information.

The recommendations for bending/straightening reinforcing bars involves bar diameter, bend diameter, heating temperature (hot-bending), and use of non-strain aging steel. The recommendations for resin grouting reinforcing bars involves embedment length, hole diameter, and type of resin.

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FOREWORD

This report consists of two volumes. The first volume reviews the Washington State Department of Transportation's specification guidelines for field bending/straightening concrete reinforcing bars, and proposes modifications to those guidelines based on current knowledge. The second volume reviews the Department's specification guidelines for resin grouting epoxy-coated concrete reinforcing bars, and suggests modifications to the guidelines for grouting based on the available information.

BENDING/STRAIGHTENING AND GROUTING CONCRETE REINFORCING STEEL: REVIEW OF WASHING TON STATE DEPARTMENT OF TRANSPORTATION'S SPECIFICATIONS AND PROPOSED MODIFICATIONS

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VOLUME ONE

BENDING/STRAIGHTENING CONCRETE REINFORCING BARS

VOLUME ONE
TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

SUMMARY

Reinforcing bars partially embedded in concrete and protruding from it are frequently subject to bending and straightening in the field. Often it is necessary to bend the protruding bars to provide clearance for construction operations. Furthermore, field bending and/or straightening is required because of incorrect fabrication or accidental bending. Whatever the cause, the success rate of bending/straightening bars in the field has been unpredictable, and cracking of the bars has been reported. Even in the absence of cracking, there has been concern over the effects of bending/straightening on the engineering properties of concrete reinforcing bars.

Presently, the Washington State Department of Transportation (WSDOT) recommends two different procedures for field bending/straightening reinforcing bars. These are cold-bending and hot-bending. Cold-bending means bending bars whose temperature is close to ambient temperature. Hot-bending means bending bars that are heated to elevated temperatures to reduce their yield strength while bending. Generally, hot-bending increases the rate of success of the bending/straightening when large bars are involved.

WSDOT initiated this research project to review its present specification guidelines for field bending/straightening reinforcing bars and to recommend modifications to those guidelines so that it can increase the success rate of field bending/straightening and minimize possible changes in the engineering properties of reinforcing steel. After the available relevant data were collected, they were analyzed and the results were compared to WSDOT's current specification guidelines. Accordingly, recommendations for improvement of those guidelines were made. Finally, a second research phase consisting of laboratory tests was suggested to verify and/or modify the recommendations of this study.

The study recommended that the WSDOT's required bend diameter/bar diameter ratios for cold-bending and hot-bending be increased for certain bar sizes. This recommendation was based on both the expected strain capacity of ASTM A615 grade 60 reinforcing steel and the expected embrittlement caused by strain aging of such steel. The most effective temperature range for hot-bending was found to be 1400 to 1500°F. That temperature range assures that temperatures in the center of the bar are above a range of 400 to 700°F, since temperatures between 400 and 700°F can cause brittle fracture during the operation. However, the WSDOT required temperature range of 1100 to 1300°F was not increased because of concerns about controlling the temperature in the field and about overheating and degrading the steel. For hot-bending, guidelines for the heating period were provided to prevent excessive heating periods from causing strain age embrittlement of the metal. Also, recommendations were made concerning the bend diameter and use of non-strain aging steel to assure the development of seismic hinges when reinforcing bars to be bent in the locations designed for such hinges.

FINDINGS

The following findings are based on the data analysis and assessments presented in the body of this report.

COLD-BENDING

1. The minimum elongation required by ASTM for A615 grade 60 steel is 9 percent for No. 6 and smaller bars, 8 percent for No. 7 and No. 8 bars, and 7 percent for No. 9 and larger bars. The WSDOT's required bend diameter of 6 bar diameters can induce a strain of about 16 percent and lower in No. 5 and smaller bars, 16.5 percent in No. 6 bars, and higher than 16.5 percent in bars larger than No. 6 bars. This finding is based on consideration of stress concentrations in the bar deformations. However, ASTM A615 grade 60 bars have usually shown a strain capacity about twice as much as the minimum elongation required.
2. When bars are bent without a bend former, the desired bend diameter may be obtained if the distance from the face of the bending tool (pipe placed over the bar) to the restraint (concrete) is controlled. A distance of 6 in. can produce a bend diameter of 10 bar diameters for No. 6 bars, and larger bend diameter for smaller bars, provided that the bend angle does not exceed 90 degrees. The bending tool should have freedom to move at the point of bending.
3. When bars are bent without a bend former and without any consideration for the distance from the face of the bending tool to the concrete (i.e., free bending), the WSDOT's required bend diameter of 6 bar diameters cannot be guaranteed if the bend angle exceeds 45 degrees.
4. After bending, bars are subject to embrittlement. This phenomenon is the result of strain aging. Increases in both the period of aging and the ambient temperature increase that embrittlement. Also, the embrittlement increases as the amount of strain induced by bending increases. For deformed bars, high local strains can

develop during bending as a result of stress concentrations in the deformations. Those local areas are the nucleation points for brittle fracture during the subsequent straightening operation.

5. When bars are bent according to the WSDOT's required bend diameter of 6 bar diameters and subsequently aged during construction, they may be successfully straightened if they are not larger than a No. 6 bar (this statement is not valid for temperatures below 41°F, in which the metal brittleness is more severe). The same logic is applicable to shop-bent bars placed at locations designed to perform as seismic hinges. Those bars strain age after shop bending and are subject to yielding in the event of a major seismic activity.

6. Bending/straightening bars results in a loss of metal ductility, even in the absence of strain aging. A reduction in the tensile elongation equal to about 25 percent has been reported for a bar of 1 1/8 in. in diameter bent to a diameter of 7 times its diameter. Aging both after bending and after straightening can further reduce the elongation.

7. Bending/straightening bars may increase their yield and ultimate strength in the presence of strain aging.

8. When a bent, aged, and straightened bar is aged a second time after the straightening, further embrittlement in the bar may occur. Theoretically, this condition can cause brittle fracture of the bar upon rebending, unless the bend diameter is at least 30 bar diameters. An implication of this theory is that, in the event of a major seismic activity, bars bent, straightened, and placed in locations designed for seismic hinges are subject to brittle fracture.

9. Concerning cyclic service loading on bridges, bending/straightening bars does not seem to affect their fatigue strength.

10. Higher rates of success in bending/straightening reinforcing bars can be obtained by controlling the content of the various alloys in the steel to assure its ductility, and

also by controlling the effects of strain aging. Non-strain aging steel has been produced in New Zealand by the addition of titanium.

HOT-BENDING

1. When steel is heated, both its yield strength and its modulus of elasticity decrease significantly. Consequently, bending/straightening a heated bar requires application of less force. This condition reduces the input energy on the bar during the operation. Also the heated bar's lower rigidity increases its impact resistance during the operation. As a result, the possibility for cracking is minimized significantly. There are clear indications that hot bending/straightening results in better prevention of bar cracking than cold bending/straightening. This is especially evident when the operation involves large bars or cyclic bending/straightening. For example, No. 11 bars have been successfully bent and straightened with a bend diameter as small as 4 bar diameters. No. 5 bars have been bent and straightened three times with a bend diameter as low as 3.5 bar diameters.
2. Hot-bending at temperatures in the range of 400 to 700°F can cause brittle fracture during the operation. Also, bending at temperatures in the range of 1100 to 1200°F may result in the same type of failure, since the temperature of the center of the bar may not exceed 700°F. The most effective temperature is within the range of 1400 to 1500°F. If the temperature is increased to 1800°F, failure of the bar may occur due to the formation of a brittle grain in the metal. Therefore, if temperature control in the field is difficult, the maximum heating temperature may be specified well below 1800°F, such as 1300°F.
3. Hot bending/straightening can result in approximately a 10 percent reduction in the ultimate strength of the steel after it cools. Hot bending/straightening can also reduce the corresponding elongation. The amount of reduction in the elongation increases as the amount of strain induced during the bending increases. Reductions

in the elongation of approximately 25 and 10 percent can be expected for bend diameters of 6 and 8 bar diameters (i.e., WSDOT's required bend diameters for hot bending), respectively.

4. Hot-bending may result in further embrittlement after the steel cools, if the bent bar is kept at elevated temperatures (i.e., 800°F or more) for an appreciable time period (e.g., 300 seconds). This embrittlement is due to the strain aging during heating. For a short time period (e.g., 30 seconds), that embrittlement may not be significant.

5. In addition to the effects of strain aging at elevated temperatures, hot-bent bars are subject to the effects of strain aging at ambient temperature both after bending and after straightening. Thus, the seismic concern of cold bending/straightening should also be valid for hot bending/straightening. This means that ASTM A615 grade 60 bars placed in locations designed as seismic hinges may be subject to failure in the event of a major seismic event, if they were hot bent and straightened during construction.

RECOMMENDATIONS FOR TENTATIVE SPECIFICATION GUIDELINES FOR BENDING/STRAIGHTENING REINFORCING BARS

The following presents recommendations for tentative specification guidelines for field bending/straightening and shop bending ASTM A615 grade 60 concrete reinforcing bars. These recommendations may be modified based on the results of supplementary laboratory research.

COLD-BENDING

1. Cold bending and straightening should be allowed only if
   - the bar is No. 6 or smaller;
   - the number of plastic strain cycles that the bar will have endured since its fabrication will not exceed one (One cycle comprises one bending and subsequent straightening. Thus, bars previously shop-bent can be straightened, but cannot be rebent.);
   - the ambient temperature (or bar temperature) is 45°F or more;
   - the bend former diameter is at least six times the bar diameter for No. 3 and No. 4 bars, and at least eight times the bar diameter for No. 5 and No. 6 bars. (This condition prevents excessive embrittlement due to strain aging and results in local strain in the deformations of about 75 to 50 percent more than the minimum elongation required by ASTM. Note that the expected elongation capacity is at least twice the minimum elongation required.); and
   - if a bend former is not used, the distance from the face of the bending tool in field bending (i.e., the pipe placed over the bar, or "hickey bar") to the restraint (i.e., the concrete) is at least 6 in. and the bend angle is limited to 90 degrees. The bending tool must have freedom to move at the point of bending.

2. Bending/straightening reinforcing bars placed in locations designed to perform as seismic hinges should not be allowed. A chemically controlled, non-strain aging steel may be used.

3. Shop bending reinforcing bars placed in locations designed to perform as seismic hinges should be limited to No. 6 and smaller bars, and the bend diameter should be at least eight times the bar diameter. Otherwise, a chemically controlled, non-strain aging steel may be used.

HOT-BENDING

1. Hot or cold bending/straightening of bars placed at locations designed to perform as seismic hinges should not be allowed. If bending/straightening of such bars is necessary, a chemically controlled, non-strain aging steel may be specified.

2. Hot-bending may be performed when cold bending is not allowed. Hot bending/straightening bars larger than No. 11 bars is not recommended.

3. The bend former diameter should not be smaller than eight times the bar diameter for No. 9 and smaller bars. The former diameter should not be smaller than 10 times the bar diameter for No. 10 and No. 11 bars.

4. The bending temperature should be 1200°F for No. 6 and smaller bars, 1250°F for No. 7 through No. 9 bars, and 1300°F for No. 10 and No. 11 bars. Also, temperatures in the range of 400 to 800°F must be strictly avoided. The latter range of temperature is possible if the bending operation is delayed and not performed immediately upon heating.

5. Heating should be performed by two heat tips simultaneously at opposite sides of the bar to assure uniform temperature throughout the thickness of the bar and to minimize the heating time. Heating time of a bent bar should not exceed the minimum time required to bring the temperature of the bar to the specified

temperature. The minimum bar length to be heated should be at least equal to the length of the bend and should be determined from the bend diameter and angle.

INSPECTION

1. Inspection of bars for cracking should not be done earlier than 24 hours after the bending operation. Experience has shown that delayed cracking may appear in bent bars after the bending operation.

PROPOSED SPECIFICATION FOR BENDING/STRAIGHTENING REINFORCING BARS

If the plans call for field bending of steel reinforcing bars, the Contractor shall bend them in keeping with the structure configuration and the plans.

Bending steel reinforcing bars partly embedded in concrete shall not be done until the Engineer has given written approval of a field-bending plan to the Contractor. Approval for such bending will be given only for bars smaller than Size No. 14.

Field bending shall not be done:

1. On bars Size No. 14 and 18,
2. When air temperature is lower than 45°F,
3. By means of hammer blows or pipe sleeves, or
4. While the bar temperature is in the range of 400 to 700°F.

In field-bending steel reinforcing bars, the Contractor shall:

1. Make the bend gradually;
2. Apply heat as described below in bending bar Sizes No. 7 through No. 11 and in bending bars Sizes No. 6 and smaller when the bars have been previously bent. Previously unbent bars of Sizes No. 6 and smaller may be bent without heating;
3. Use a bending tool equipped with a bending diameter as follows:

	Bend Diameter/Bar Diameter Ratio*	
Bar Size	Heat Not Applied	Heat Applied
No. 3, No. 4	6	8
No. 5, No. 6	8	8
No. 7, No. 8, No. 9	Not permitted	8
No. 10, No. 11	Not permitted	10

4. Limit any bend to these maximums: 135 degrees for bars smaller than Size No. 9, and 90 degrees for bars Size No. 9 through No. 11; and
5. Straighten by moving a hickey bar (if used) progressively around the bend.

*Bend former should turn freely

In applying heat for field-bending steel reinforcing bars, the Contractor shall:

1. Use a method that will avoid damage to the concrete;
2. Insulate the concrete within 6 inches of the heated bar area;
3. Ensure by means of temperature-indicating crayons, or other suitable means, that the steel temperature never exceeds the maximum temperature shown below:

	Temperature (°F)	
<u>Bar Size</u>	<u>Minimum</u>	<u>Maximum</u>
No. 3, No. 4, No. 5, No. 6	1,100	1,200
No. 7, No. 8, No. 9	1,150	1,250
No. 10, No. 11	1,200	1,300

4. Maintain the steel temperature within the required range shown above during the entire bending process;
5. Apply two heat tips simultaneously at opposite sides of bars larger than Size No. 6 to assure a uniform temperature throughout the thickness of the bar. For Size No. 6 and smaller bars apply two heat tips, if necessary;
6. Apply the heat for a long enough time that within the bend area the entire thickness of the bar -- including its center -- reaches the required temperature;
7. Bend immediately after the required temperature has been reached;
8. Heat at least as much of the bar as indicated below:

	Heated Length, Based on Bar Diameter Bend Angle, Degrees		
<u>Bar Size</u>	<u>45°</u>	<u>90°</u>	<u>135°</u>
No. 3 through No. 8	8d	12d	15d
No. 9	8d	12d	Not permitted
No. 10 and No. 11	9d	14d	Not permitted

9. Locate the heated section of the bar to include the entire bending length; and
10. Never cool bars artificially with water, forced air, or other means.

INTRODUCTION

Reinforcing bars partially embedded in concrete and protruding from it are frequently subject to bending and straightening in the field. Often it is necessary to bend the protruding bars to provide clearance for construction operations. Another alternative is to cut and splice such bars. However, this alternative is costly and not preferred. Recognizing construction requirements and cost effectiveness, Washington State Department of Transportation (WSDOT) designers specify that certain bars be bent in the field to meet construction needs and subsequently be straightened. Furthermore, field bending and/or straightening is often required by incorrect fabrication or accidental bending.

Whatever the cause, the success rate of bending/straightening bars in the field has been unpredictable, and cracking and breaking of the bars have been reported. Even in the absence of the cracking, WSDOT has been concerned over the effects of bending/straightening on the engineering properties of reinforcing bars. The present procedures recommended by WSDOT for bending bars in the field are in the early stages of development and need to be verified and/or modified. Additionally those procedures need fine tuning before standard specifications are developed.

OBJECTIVES

WSDOT initiated this research project with the objectives described below:

- to review WSDOT's current procedures for bending/straightening reinforcing bars in the field, and to compare those procedures with relevant research findings and performance information, and
- to develop tentative guidelines for WSDOT designers and field engineers for increasing the success rate of bending/straightening bars in the field.

RESEARCH APPROACH

After the WSDOT's current specification guidelines for field bending/straightening reinforcing bars were reviewed, information was gathered from U.S. and foreign sources on the procedures of bending/straightening reinforcing bars, as well as the performance of the bars during and after bending. That information was then analyzed for evidence that bars may crack during the bending/straightening operation and that the operation may render the engineering properties of the bars unsuitable for reinforcing the concrete.

In addition, a reinforcing bar manufacturing plant and its bar bending shop were visited, and WSDOT field inspectors were interviewed to further review related procedures, performance, and problems. Subsequently, tentative recommendations for specification guidelines were made for WSDOT's consideration. Finally, a second research phase consisting of laboratory tests was suggested to verify and/or modify the recommendations of this work.

REVIEW OF WSDOT'S SPECIFICATION GUIDELINES

WSDOT specifies two different procedures for bending/straightening reinforcing bars in the field. These are

- cold-bending, and
- hot-bending

The elements that influence the choice of a bending procedure are

- bar size,
- number of plastic strain cycles endured by the bar, and
- ambient temperature.

COLD-BENDING

Cold-bending means bending bars whose temperature is the same as the ambient temperature. WSDOT specifies cold-bending only if

- the bar is No. 6 or smaller,
- the number of plastic strain cycles that the bar will have endured since its fabrication will be one or less (This means that the bar has not been previously shop- or field-bent. If the latter type of bars are straightened and subsequently rebent, then the number of plastic strain cycles after the operation will be 1-1/2), and
- the ambient temperature is above 40°F.

WSDOT recommends control over the radius of bend to avoid sharp curvatures and potential for cracking. A bending tool providing a bending diameter of 6-bar diameters is used. Bending is performed in a gradual manner. Impact bending by hammer blows is not permitted.

HOT-BENDING

Hot-bending means bending bars that are heated for the bending operation. Hot-bending may be performed when cold-bending is not permitted.

Hot bending requires steel temperatures of 1100 to 1300°F, but in no case should the temperature exceed 1300°F, as measured by temperature indicating crayons or other approved means. Also, temperatures in the 400 to 700°F range must be avoided while bending. If the heated zone is within 6 in. of the concrete, insulation is applied to protect the concrete.

Heating is performed by two heat tips simultaneously applied at opposite sides of the reinforcing bar to assure a uniform temperature throughout the thickness of the bar. It is critical that the center of the bar reaches the required temperature. WSDOT has recommended the following heat tips in the past:

- Torchweld No. 36 with an oxygen-acetylene mixture, and
- Victor No. 8 MFA heating nozzle with an oxygen-propane mixture.

The bending diameter is 6-bar diameters for No. 3 through No. 8 bars, and 8-bar diameters for No. 9 through No. 11 bars. The heated length is the curved portion of the bar but not less than a minimum length, as given in Table 1. The heated length is located so that it includes the entire bending length. After bending, the heated bar must not be artificially cooled by means such as water or forced air.

INSPECTION AND REPAIR

All cold-bent or hot-bent bars, if cracked or broken during the bending/straightening operation, are replaced with a mechanical splice. Absence of cracking in the bars indicates a satisfactory bending/straightening operation.

Table 1. WSDOT's Recommended Minimum Bar Heated Length

Bend Angle (Degrees)	Minimum Heated Length (inches)	
	No. 3 thru No. 8	No. 9 thru No. 11
45	6d	8d
90	9d	12d
135	12d	not permitted

d = bar diameter

INTERPRETATION OF INFORMATION, APPRAISAL, AND APPLICATION

Bending/straightening reinforcing bars may result in two types of distress in bars. The first state of distress is degradation in the steel without any sign of cracking. Degradation means changes in the engineering properties of the steel, such as a reduction in ductility and strength. The second state of distress is the limit state, which is cracking of steel. Although the occurrence of degradation is far less serious than the occurrence of cracking, degradation of a bar can lead to its cracking during the service period of the structure, if that degradation affects the bar's fatigue and seismic strength. This chapter discusses the effects of various factors in cold- and hot-bending on the occurrence of degradation and cracking in reinforcing bars.

COLD-BENDING

In cold-bending, the following factors can cause cracking in the steel during the operation, can affect the steel's strength, and/or can result in embrittlement in the steel:

- strain,
- strain-aging,
- cyclic strain,
- ambient temperature, and
- chemical composition of the steel.

Bars can be visually inspected for cracking, and they can be tested in tension to document changes in yield and ultimate strength. There is no direct method to measure embrittlement in steel. The engineering properties that indicate embrittlement in steel include the following:

- elongation, or the ultimate strain (determined from a stress-strain curve),
- impact resistance, or fracture energy (determined from the Charpy impact test), and

- ductile-brittle transition temperature (determined from the Charpy impact test conducted at different temperatures).

Also, a reduction in sectional area after the bar tension failure indicates embrittlement. Less reduction in area corresponds to higher embrittlement.

Strain

Strain is a function of bend diameter and bar diameter. Tensile strains induced on the outside of a bend can be approximated from the following equation:

$$E = \frac{100}{D/d + 1} \qquad \text{(Equation 1)}$$

Where:

E = percent strain

d = bar diameter, and

D = bend inside diameter.

Equation 1 indicates that the tensile strain induced by bending increases as the bar diameter increases and the bend diameter decreases.

Generally, WSDOT specifies that reinforcing bars for concrete conform to the requirements of ASTM designation A 615 (Billet Steel Bars for Concrete Reinforcement) grade 60. Table 2 gives ASTM's minimum tensile requirements for A615 reinforcing steel. As shown in the table, the minimum elongation (ultimate strain) requirements for A615 grade 60 bars are from 7 percent to 9 percent, depending on the bar size. Note that usually, strain capacities about two times those shown in Table 2 are possible. From Equation 1 and the minimum strain requirements in Table 2, Table 3 provides minimum bend diameter/bar diameter ratios for cold-bending to avoid the ultimate strain and cracking of bars. According to Table 3, the WSDOT's specified bend diameter/bar diameter ratio of 6 does not guarantee the prevention of ultimate strain even for No. 3 bars. However, considering actual strain capacities, that ratio guarantees the prevention of ultimate strain for bars as large as No. 18 bars.

Table 2. ASTM Tensile Requirements for A615 Reinforcing Bars

	Grade 40[a]	Grade 60
Tensile strength, min. (psi)	70,000	90,000
Yield strength, min. (psi)	40,000	60,000
Elongation in 8 in., min. (%)		
Bar no.		
3	11	9
4, 5, 6	12	9
7	---	8
8	---	8
9	---	7
10	---	7
11	---	7
14, 18	---	7

[a] Grade 40 bars are furnished only in sizes 3 through 6

Table 3. Minimum Bend Diameter/Bar Diameter Ratios Satisfying the ASTM Minimum Elongation Requirements for A615 Grade 60 Bars

Bar no.	Bend diameter (1) / Bar diameter	Bend Radius (1) / Bar diameter
3	10	5
4, 5, 6	10	5
7	11.5	6
8	11.5	6
9	13	6.5
10	13	6.5
11	13	6.5
14, 18	13	6.5

(1) Inside diameter or radius

Bend orientation

Equation 1 ignores the presence of deformations in reinforcing bars. Bend orientation affects the strain induced in deformed bars. If the bend is oriented with the longitudinal rib at the extreme fiber of bending (bending around the strong axis), higher strains are induced than those given by Equation 1. On the other hand, if the bend is oriented with the longitudinal rib at the neutral axis (bending around the weak axis), lower strains are induced than those provided by Equation 1. For ASTM A615 bars, the height of the deformations increases from 0.015 in. for No. 3 bars to 0.102 in. for No. 18 bars. However, incorporating the latter values of the deformation height in Equation 1 does not change the strain significantly.

A Concrete Reinforcing Steel Institute (CRSI) laboratory study found that for some No. 11 bars, the minimum bend diameter necessary to avoid cracking while straightening was about 50 percent less for weak axis bending than for strong axis bending. (Ref. 1) However, certain bars showed improvement in straightening while bending around the strong axis. It appeared that the performance was related to the geometry of the notch both at the intersection of the transverse deformation to the longitudinal rib and at the intersection of the transverse deformation to the body of the bar. When the intersection of the transverse deformation to the longitudinal rib was less sharp than the intersection of the transverse deformation to the body of the bar, cracking was less likely in bends around the strong axis due to reduced stress concentration effects. Also of concern is deformations caused by manufacturers' markings on the bars. The role of bar deformation on stress and strain concentration will be covered later in conjunction with strain-aging effects.

Bend angle

Equation 1 indicates that the strain is independent of the bend angle. However, if the bend diameter is not controlled (i.e., free bending), the bend angle can affect the bend diameter and the induced strain. Higher angles result in smaller bend diameters, and they consequently induce higher strains.

When bars are not bent around a pin with the desired bend diameter, the bend diameter obtained will be a function of the distance (L), which is the length from the restraint (i.e., the concrete) to the free end of the bar (i.e., the face of the pipe placed on the bar to pull and bend the bar). Figure 1 illustrates the distance (L). Naturally, a smaller L results in a smaller bend diameter. When the distance L is controlled, bend diameter can be approximated from the following relation, if a uniform curvature is assumed for the overall bend length:

$$D = \frac{360L}{\pi A} \qquad \text{(Equation 2)}$$

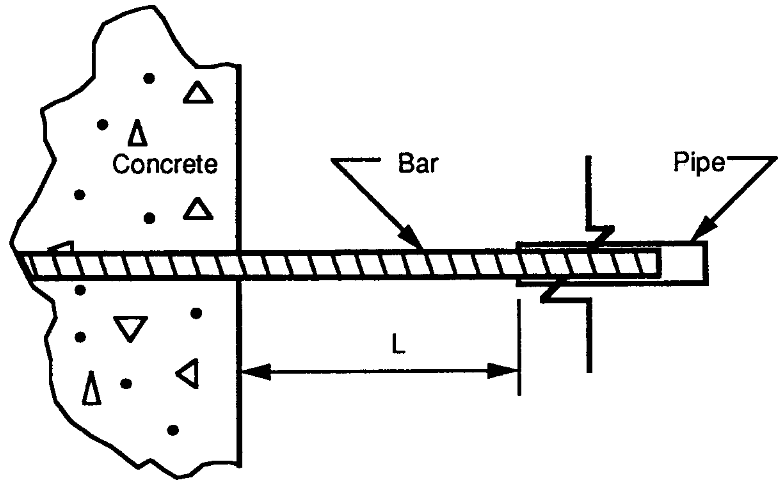
Where:

D = bend diameter, and

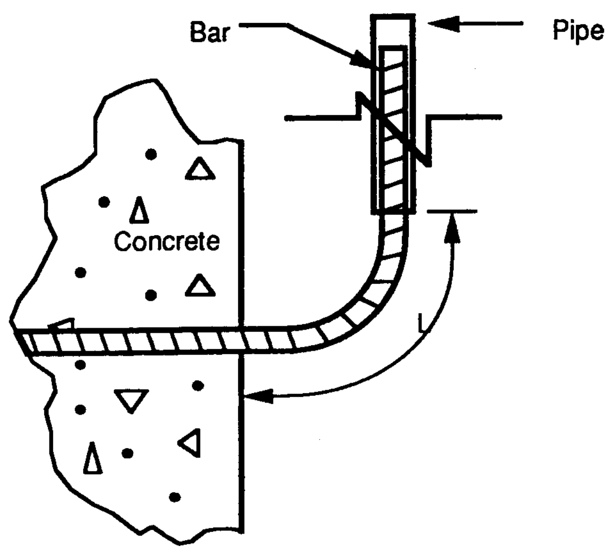
A = bend angle in degrees

Lalik and Cusick, in their laboratory tests, showed that for No. 8 bars and a bend angle of 90 degrees, a bend diameter of 8 in. (8 bar diameters) was obtained for L as small as 6 in. (Ref. 2) Using Equation 2, for the latter condition the bend diameter would be 7.6 in., which is very close to the bend diameter of 8 in. actually measured. This analysis indicates that the bend diameter can be controlled for bend angles up to 90 degrees, if there is a control on the distance (L). According to Equation 2, when a pin is not used, a distance (L) of at least 4 in. can guarantee the WSDOT requirement for cold bending No. 6 and smaller bars (i.e., a bend diameter of 6 bar diameters), provided that the bend angle does not exceed 90 degrees. Also, it is important that the bending tool is free to move at the bending point.

However, in the field, the distance (L) may not be controlled. The CRSI laboratory research (Ref. 1) showed that when the bending tool (i.e., the pipe) was located as close as possible to the restraint (i.e., L ~ 0), the bend diameter decreased with the increasing bend angle. For 15-degree bends and for No. 11, No. 8, and No. 5 bars, the bend diameter was approximately 20 in. This is equal to bend diameter/bar diameter ratios of approximately



a: Before Bending



b: After Bending

Figure 1. Illustration of Bending a Bar with a Controlled Distance from the Restraint

15 for No. 11 bars, 20 for No. 8 bars, and 32 for No. 5 bars. At a bend angle of 45 degrees, the bend diameter was approximately 8 bar diameters for No. 11 bars, and 6 bar diameters for No. 8 and No. 5 bars. The bend diameter continued to decrease with an increase in bend angle until its minimum was reached at a bend angle of 90 degrees. Beyond 90 degrees, the change in bend diameter was not significant. The minimum bend diameter achieved was approximately 4 bar diameters for No. 11 bars, and 3 bar diameter for No. 8 and No. 5 bars.

A comparison of the values of the bend diameter/bar diameter ratios obtained in the CRSI experiments with free bending (Ref. 1) with WSDOT's requirement for cold-bending (i.e., with a bend diameter of 6 bar diameters) indicates that free bending of bars in the field without any consideration to the bending length (i.e., the distance (L) in Figure 1) may satisfy WSDOT's requirements only if the bend angle does not exceed 45 degrees.

Strain Aging

Reinforcing bars bent in the field are not straightened immediately. Generally, some time lapse occurs between the initial bending of the bar and the straightening operation. A number of steel properties may change during this period due to the diffusion of interstitial carbon and nitrogen atoms to the dislocation sites in the plastic strained steel. This phenomenon is called "strain aging." Factors that affect changes in the steel properties include the following:

- the extent of the plastic strain,
- the age of the plastic strain, and
- the ambient temperature.

Increases in the magnitude of these three factors can cause more changes in the steel's properties.

Embrittlement

One effect of strain aging is steel embrittlement. Erasmus has found the relation between increases in the Charpy brittle/ductile transition temperature of New Zealand normal reinforcing bars of grade 275 (~40 ksi yield) and grade 380 (~55 ksi yield) and pre-strain, as shown in Figure 2. (Ref. 3) Those bars were aged at 212°F for 3 hours, which is equal to 6 months of aging at 70°F. (Ref. 4) The increase in the transition temperature was found to be the same for the both bar grades (Figure 2).

Table 4 gives the chemical compositions of the latter two grades of the New Zealand Bars (Ref. 3) as well as the chemical compositions of ASTM A615 grade 60 bars supplied by three sources in the United States. (Ref. 1) Table 4 indicates that the chemical properties of the ASTM grade 60 bars are about the same as those of the New Zealand grade 380 bars and not much different from the chemical properties of the New Zealand grade 275 bars. Thus, in the absence of more accurate laboratory test data, Figure 2 may also represent the increase in the Charpy transition temperature in ASTM A615 grade 60 bars as a function of pre-strain, when the bars are subject to strain aging.

Safe bend diameter to prevent brittle cracking

On the basis of limited field and laboratory information, Erasmus found that for a bent bar, a Charpy transition temperature of 127°F, caused by strain aging, is the maximum temperature that will guarantee the integrity of the bar against a brittle cracking during the succeeding straightening operation in ambient temperatures as low as 41°F. (Ref. 3) Taking 127°F as the reference fracture transition temperature, Erasmus calculated the safe bend former diameter for deformed New Zealand reinforcing bars. (Ref. 3) The following is an adaptation of the Erasmus' calculations to determine the safe bend former diameter for deformed ASTM A615 grade 60 bars to avoid their brittle cracking after strain aging and during the straightening operation.

Table 5 provides suggested Charpy transition temperatures for New Zealand grade 380 deformed bars of different sizes. (Ref. 3) Since the chemical and mechanical

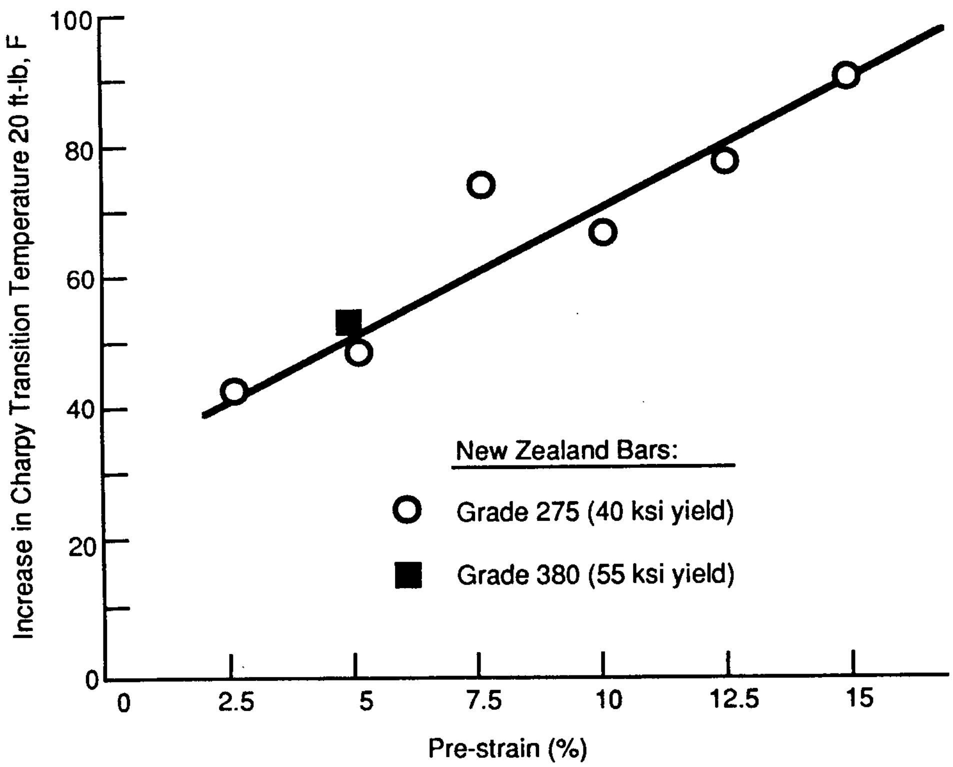


Figure 2. The Effect of Tensile Prestrain on the Increase in the Charpy Transition Temperature, 20 ft-lb Energy Level (Adapted from Ref. 3)

Table 4. Comparison of Chemical Analysis of Selected U.S. and New Zealand Deformed Reinforcing Bars (Adapted from References 1 and 3)

		New Zealand Bars						U.S. Bars					
		Grade 275 (40 ksi yield)			Grade 380 (55 ksi yield)			ASTM A 615 Grade 60 (60 ksi yield)					
								Supplier A		Supplier B			Supplier C
		16 mm (no. 5)	22 mm (no. 7)	28 mm (no. 9)	16 mm (no. 5)	22 mm (no. 7)	28 mm (no. 9)	(2) no. 11	(2) no. 11	no. 5	no. 8	no. 11	no. 11
Chemical Composition (%)	C	0.22	0.21	0.23	0.34	0.38	0.38	0.34	0.36	0.43	0.39	0.42	0.37
	Mn	0.51	0.51	0.40	1.16	1.06	1.15	1.01	0.91	1.27	1.19	1.29	1.06
	P			0.010 (1)				0.016	0.0009	0.012	0.010	0.008	0.020
	S			0.030 (1)				0.033	0.035	0.035	0.021	0.041	0.019
	Si			0.09 (1)				0.27	0.24	0.079	0.04	0.095	0.23

(1) Plain bars
(2) Different heats

Table 5. Suggested Charpy Transition Temperatures for New Zealand Grade 380 and U.S. Grade 60 Reinforcing Bars (20 ft-lb energy) and Calculated Safe Bend Diameter/Bar Diameter Ratios (Adapted from Ref. 3)

New Zealand Bar Diameter (mm)	Approximate Equivalent (US bar no.)	Charpy Transition Temperature (F)	Difference in Transition Temp. from Reference Temp. of 127°F	Corresponding Pre-Strain %[1]	Safe Bend/ Bar Diameter Ratio[2]
10	3	5	122	23	3.5
12	4	9	118	22	4.
16	5	19	108	19	5.
20	6	25	102	18	5.5
22	7	28	99	17	6.
24	8	34	93	16	7.5
28	9	41	86	14	9.5
32	10	50	77	12	14.
34	11	54	73	11	16
40	13[3]	64	63	9	30

(1) From Figure 2 or from equation:

$$\text{Pre-strain } \% = \frac{5(\text{change in Transition Temp., °F}) - 135}{21}$$

(2) From equation:

$$\text{Pre-strain } \% = \frac{100}{\frac{D}{d} + 1} + \frac{2.19 \times (\text{bar no.})}{\frac{D}{d}} + 0.63(\text{bar no.}) - 3$$

(3) Bar not manufactured in U.S.

properties of the latter bars and those of grade 60 ASTM A615 bars are about the same, Table 5 may also represent grade 60 bars in the absence of more accurate test data. Included in Table 5 are the differences in Charpy transition temperatures from the reference temperature of 127°F. The pre-strains causing those temperature differences, in the presence of strain aging, are also included in Table 5. Those pre-strains were obtained using the following equation, which was derived in this work from Figure 2:

$$\text{Percent pre-strain} = \frac{5\,(\text{increase in Transition Temperature, F})-135}{21} \qquad \text{(Equation 3)}$$

With the values of pre-strain known, safe bend diameter/bar diameter ratios can be obtained from Equation 1. However, Equation 1 gives strains for plain bars. For deformed bars, higher local plastic strains can develop on the inner radius surface due to the stress concentration effects of the flattening of the deformations. Those local areas are the nucleation points for brittle fracture when the bend is opened out. The following empirical equation gives localized plastic strains for deformed bars:

$$E_{def} = E + \frac{2.19\,(\text{bar no.})}{D/d} + 0.63\,(\text{bar no.})-3 \qquad \text{(Equation 4)}$$

in which E_{def} is the percent of strain for deformed bars and E is the percent of strain for plain bars and is obtained with sufficient accuracy from Equation 1, although Erasmus presents a more sophisticated procedure to obtain E.

By substituting the pre-strain values of Table 5 for E_{def} in Equation 4, the bend diameter/bar diameter ratios to avoid brittle cracking of the steel can be obtained. These are included in Table 5. A comparison of Table 5 with Table 3 indicates that for No. 9 and smaller bars, the development of ultimate strain is most likely the cause of cracking in bars, whereas, for bars larger than No. 9, brittle fracture due to strain aging can precede development of the ultimate strain. The data in Table 5 led to the conclusion that WSDOT's required bend diameter of 6 bar diameters for cold bending cannot prevent brittle cracking caused by strain aging in No. 7 and larger bars.

Cyclic Strain

A bending operation and the following straightening operation comprise one plastic strain cycle. Reinforcing bars can be subject to more than one strain cycle in the field. If shop-bent bars are straightened and later bent to the original configuration, the overall operation comprises 1-1/2 strain cycles.

When cycles of strain are applied to a steel sample, loading to yield in one direction significantly reduces the subsequent yield strength in the opposite direction. This phenomenon is shown in Figure 3. (Ref. 5) On the other hand, if after the application of strain in each direction the steel is allowed to age, the material hardens and yields at a new stress level when strained in the opposite direction. This new yield strength is above that of the unaged yield strength and is also above the original yield strength. (Ref. 4) Regardless of aging, cyclic cold bending can increase the ultimate strength, but it can cause embrittlement in the steel. (Ref. 1, 2, 4, 6) Data presented in Ref. 4 indicates that 1-1/8 in. diameter bars lost about 25 percent of their tensile elongation when they were bent 90 degrees around an 8-in. diameter former (a bend diameter of 7 bar diameters, or 13 percent strain), straightened, and tested in tension immediately. As a general rule, the sum of the tension and compression strains (e.g., the strain from B to C in Figure 3) should not exceed the ultimate strain of the material for unidirectional tension. This is because of the cumulative effects of straining on the ultimate strain.

Embrittlement caused by strain aging

Plastic bending of steel in one direction and aging of the corresponding strain cause embrittlement in the steel, as discussed earlier. If that steel is subsequently straightened and allowed to age one more time, aging of the straightened steel can add to the embrittlement due to the accumulation of strain aging. Tension test stress-strain curves for bent/straightened reinforcing bars with no aging, aging after straightening only, and aging after both bending and straightening clearly show further reduction in ductility after application of aging to each stage of plastic strain. (Ref. 4)

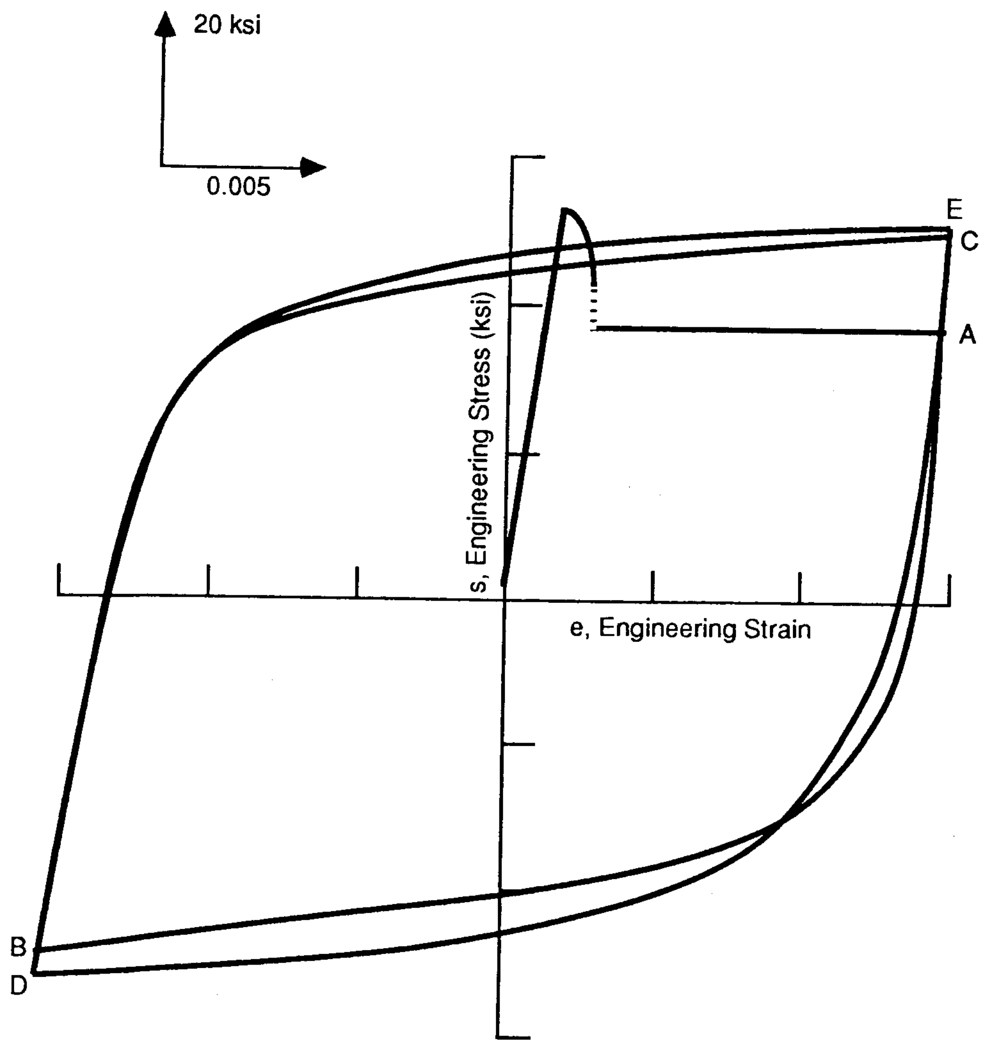


Figure 3. First Five Reversals (4-1/2 Cycles) for a Constant Amplitude Strain Controlled Test for A36 Steel (ref. 4)

An increase in the Charpy transition temperature indicates the occurrence of embrittlement in steel. Generally, each bending operation of bars in the field involves the same strain level and aging condition. Thus, it may be assumed that an increase in the transition temperature will be the same for each bending operation. This concept can be applied to Table 5 to determine safe bend diameter/bar diameter ratios for ASTM A615 grade 60 bars when they are bent in the field 1-1/2 cycles rather than one cycle. In case of 1-1/2 bending cycles, the difference in the Charpy transition temperature from the reference temperature of 127°F (Table 5) corresponds to two aging cycles rather than one aging cycle. Therefore, the values of the difference in transition temperature from the reference temperature should be divided by two before they are used in Equation 3 (or Figure 3). This procedure gives a safe bend diameter for a No. 3 bar of about 30 bar diameters to avoid brittle fracture when 1-1/2 bending cycles are involved (e.g., in the straightening and rebending of shop-bent bars). Obviously, in this case the safe bend diameter/bar diameter ratios for bars larger than No. 3 bars will be greater than 30. This finding supports WSDOT's recommendation for hot-bending when the number of plastic strain cycles is more than one.

<u>Seismic concerns</u>

Plastic hinges can form at the ends of reinforced concrete columns during a major seismic event. Generally, in concrete bridge construction, the yield capacity of a column is less than that of the beam framing into that column. In that construction and in simple span construction, plastic hinges can form at the ends of columns. The availability of sufficient ductility in those locations is important, since column plastic deformations reduce the intensity of seismic reactions in elements of the substructure. If column reinforcing bars at the locations designed to provide seismic hinges are bent, aged, and straightened during construction, the straightened bars will also age after construction and before the occurrence of an earthquake. Thus, the occurrence of a seismic hinge results in the development of at least 1-1/2 cycles of bending and two strain aging stages. As discussed earlier, under these circumstances ASTM A615 grade 60 bars may only be cold bent and straightened at a diameter of 30 bar diameters or more (depending on bar size) to avoid their brittle failure.

Another concern is the strain aging of shop-bent bars placed at the locations designed to perform as seismic hinges. For those bars, the occurrence of a seismic hinge results in development of at least one complete cycle of bending and one strain aging stage. Bars in the shop are generally bent to a diameter of 6 to 10 bar diameters, depending on their size. However, the bend diameter should follow the diameters in Table 5 to prevent brittle fracture of the reinforcing steel in the event of a major seismic event.

Fatigue concerns

During a bridge's service period, the reinforcing bars are subject to non-plastic cyclic strain. Embrittlement in deformed tension bars may create the potential for their premature fatigue fracture. For the repair of damaged steel bridge members and the effects of that repair on metal fatigue, NCHRP Report 271 recommends that member areas that have a nominal strain more than 15 times the yield point strain (a strain of approximately 3 percent for grade 60 bars, or the equivalent of a bend diameter/bar diameter ratio of 32) should not be straightened when these strains occur at severe fatigue critical areas. (Ref. 8) Severe fatigue critical details are defined as AASHTO stress categories lower than Category C.

For fatigue, AASHTO Standard Specifications for Highway Bridges limits the range between the maximum tensile stress and minimum stress in straight reinforcement as follows:

$$F_f = 21 - 0.33\, F_{min} + 8(r/h) \qquad \text{(Equation 5)}$$

Where:

F_f = stress range in ksi

F_{min} = algebraic minimum stress level, tension positive and compression negative, in ksi r/h = ratio of base radius to height of rolled-on transverse deformation; when the actual value is not known use 0.3.

On the other hand, for steel bridge members, AASHTO stress categories lower than Category C correspond to stress range limitations equal to or less than 7 ksi for 2,000,000 load repetitions. Substituting a stress range limitation of 7 ksi for F_f in Equation 5 makes F_{min} equal to 50 ksi, whereas F_{min} for bridge reinforcing steel is about 15 ksi or less for grade 60 steel. This analysis implies that the NCHRP considerations for severe steel fatigue critical areas do not apply to bridge reinforcing bars.

Temperature

Laboratory tests have indicated that the rate of success of bending/straightening reinforcing bars definitely decreases when the ambient temperature decreases from 70°F to 30°F. (Ref. 1) If the ambient temperature falls below the steel's Charpy ductile/brittle transition temperature, the possibility of brittle fracture exists when the bars are bent, depending on the magnitude of the ambient temperature and the rate of strain applied. Limited data gathered from the field (Ref. 3) indicate that for the strain rate generally applied in the field, a Charpy transition temperature of 127°F may assure safe bending at ambient temperatures as low as 41°F. Table 5 was based on the former temperature to provide a safe bend diameter against brittle fracture. Thus, the values of the bend diameter in Table 5 cannot assure safe bending in ambient temperatures below 41°F.

Chemical Composition

ASTM Standard Specifications for A615 reinforcing steel describes the chemical requirements as follows:

> "5.1 An analysis of each heat of steel shall be made by the manufacturer from test samples taken preferably during the pouring of the heats. The percentage of carbon, manganese, phosphorus, and sulfur, shall be determined. The phosphorus content thus determined shall not exceed 0.06%.

> 5.2 The chemical composition thus determined shall be reported on request to the purchaser or his representative.
>
> 5.3 An analysis may be made by the purchaser from finished bars. The phosphorus content thus determined shall not exceed that specified in 5.1 by more than 25%."

In addition to the ASTM chemical requirements, WSDOT Standard Specifications for Welding Reinforcing Steel requires that

> "Steel which is to be welded shall have a maximum carbon equivalent of 0.65 percent. The carbon equivalent shall be determined by the following formula:
>
> Carbon Equivalent = percent Carbon + percent Manganese/6 + percent Chromium/10 - percent Molybdenum/50 - percent Vandanium/10 + percent Copper/40 + percent Nickel/20.
>
> The carbon shall not exceed 0.45 percent and the manganese shall not exceed 1.30 percent."

As evidenced from the preceding descriptions, generally the chemical requirements of the reinforcing bars are not specified in detail. Reinforcing steel contains at least 12 different alloys. The content of these elements in steel can seriously affect the strength and brittleness. The following discussion, summarized from NCHRP Report 272, describes the effects of the more common alloy elements on strength and brittleness. (Ref. 7)

Strength

Figure 4 depicts the effects of the alloys on the yield and ultimate tensile strength. As shown in the figure, while the addition of carbon and nitrogen significantly increases the strength, the addition of chromium weakens the steel.

Brittleness

Carbon (C). The addition of carbon impairs toughness. (Toughness, as opposed to brittleness, is defined as the ability to absorb energy by undergoing plastic deformation prior to fracture. The measure of this ability at a high rate of loading is called impact strength.) The impairment of toughness is indicated by an increase in the steel's

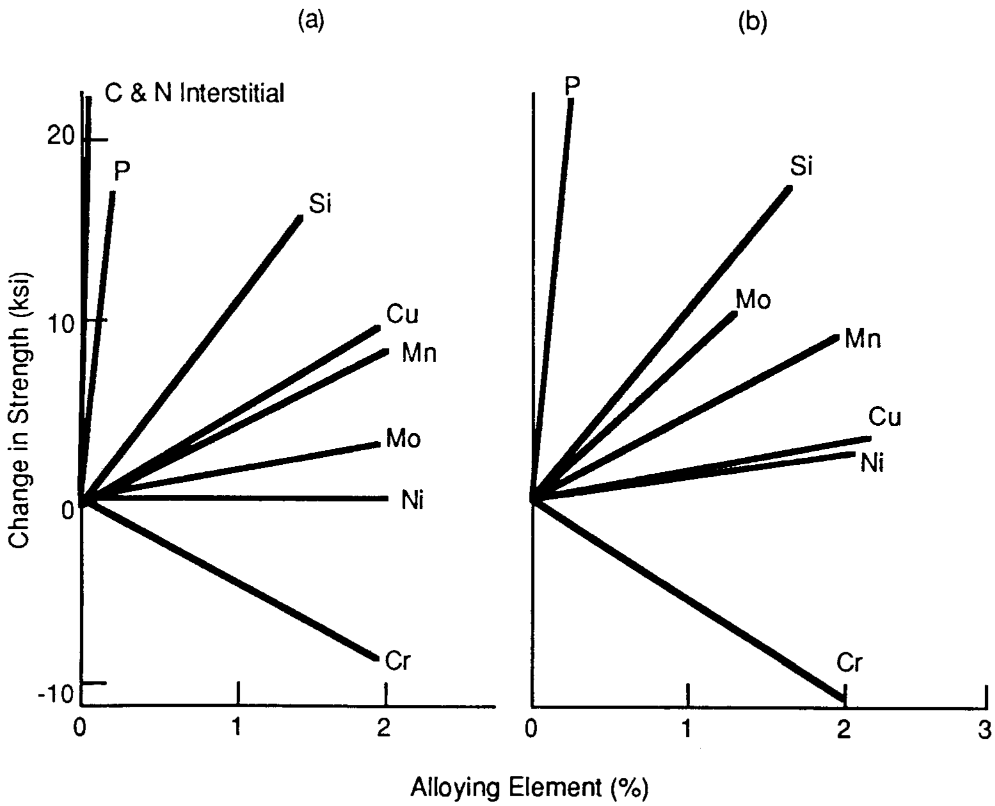


Figure 4. Effects of Solid Solution Alloys on Change in
(a) Lower Yield Point, and (b) Ultimate Tensile Strength
(Pickering, 1976) (adapted from ref. 7)

Charpy (impact) transition temperature. Structural steels generally have a carbon content below 0.8 percent.

Copper (Cu). Toughness is not affected by copper contents below 0.75 percent.

Manganese (Mn). Manganese improves toughness, but higher ratios of manganese to carbon are harmful to toughness.

Molybdenum (Mo). Molybdenum tends to impair toughness even when present in relatively small amounts.

Nitrogen (N). Although nitrogen strengthens steel, it increases the Charpy transition temperature by 50°F for each 2 ksi increase in yield strength.

Nickel (Ni). Nickel effectively lowers the transition temperature.

Phosphorus (P). Phosphorus markedly increases the transition temperature.

Sulfur (S). Sulfur is very detrimental to toughness. The reduction of sulfur lowers the transition temperature.

Silicon (Si). Silicon in amounts up to 0.30 percent lowers the transition temperature. Higher amounts reduce ductility.

Titanium (Ti). Titanium is added to ensure toughness. (The addition of titanium in the range of 0.020 to 0.035 percent reduces the rolled Charpy transition temperature and it also mitigates strain-age embrittlement and the corresponding increase in the transition temperature. (Ref. 3))

HOT-BENDING

When steel is heated, its yield strength decreases. About 75 percent of the room temperature yield strength of hot rolled steel is lost at about 1300°F. (Ref.8, 9) Generally, after steel is heated to above 1300°F, the rate of decrease in yield strength is not significant. Also, steel has a lower modulus of elasticity at higher temperatures. The modulus of elasticity decreases linearly to 900°F. (Ref. 9) At 900°F the modulus of elasticity is about

80 percent of the room temperature modulus of elasticity. Beyond that temperature the modulus of elasticity decreases at a much higher rate.

Consequently, bending or straightening heated steel requires application of less force and input energy. Also the heated bar's lower rigidity contributes to its impact resistance. There are clear indications that hot-bending results in more successful prevention of bar cracking when higher magnitudes of strain or higher numbers of strain cycles are applied. This has made the application of hot-bending especially attractive for large diameter bars and where bars are subject to more than one cycle of plastic strain. However, hot-bending raises concerns about the adverse effects of heating the bars on their engineering properties, such as loss in strength and formation of embrittlement.

Rate of Success in Bending

Laboratory tests on No. 11 grade 60 bars have found that it is safe to bend/straighten bars with diameters as small as 4 bar diameters at 1500°F. (Ref. 1) For the same bars at 70°F, the safe bending/straightening diameter was about 9 bar diameters. When the temperature was decreased to 30°F for cold bending bars of 9 bar diameters, the rate of success was only 50 percent (i.e., half of the bars broke during the straightening). Note that the bars in those tests were not aged prior to straightening. Thus, the safe bend diameter reported for cold-bending can be higher when the effects of strain-aging are considered.

In cyclic bending and straightening, significant improvement in the prevention of bar cracking can be achieved by hot bending. Tests have shown that for No. 5 bars, three cycles of bending and straightening can be achieved with a bend diameter/bar diameter ratio as low as 3.5. (Ref. 6) The same bars, when bent cold under the same conditions, can not endure more than one cycle.

Effects of temperature on rate of success

Hot-bending at temperatures between 400 to 700°F can cause brittle fracture. This has been confirmed through tests conducted at the California Department of Transportation. (Ref. 6) This temperature range is called the blue-brittleness range, and it should be avoided while bending bars. The same tests have indicated that heating bars to 1100 to 1200°F may also result in failure in bending/straightening, since the bars may not be heated long enough to bring the temperature of their center above the blue-brittleness range. The results of these tests (Ref. 6) and the results from other research (Ref. 1) indicates that the most effective temperature for hot-bending is within the range of 1400 to 1500°F. When the temperature is increased to 1800°F bars can fail possibly due to the formation of a brittle grain structure at the bar surface. The results of laboratory tests have indicated that there is a significant reduction in the rate of bending/straightening success when bars, varying in size from No. 3 to No. 10, are heated to 1800°F during the operation. (Ref. 6) The latter tests included bend diameters of up to 11 bar diameters.

Effects of Heating on Steel Properties

Although hot-bending may successfully bend and straighten reinforcing bars, changes in the steel properties may affect their performance during the structure's service period. Reinforcing bars have low amounts of carbon and are made of low alloy steel. Generally, the material properties of non-strained steel of this nature should not show significant changes when the heating temperature is below 1333°F, the steel's phase change temperature. The following discusses the effects of heating strained reinforcing steel on its strength and embrittlement.

Strength

Tests have shown that generally, after hot-bending/straightening (1400 to 1500°F) when the steel has cooled to the ambient temperature, it has essentially the same yield strength as the original material's yield strength. However, after hot-bending/straightening there is an approximately 10 percent to 12 percent reduction in the ultimate strength. (Ref. 1, 6) This finding is based on tests conducted on No. 11 and No. 8 bars.

Embrittlement

The amount of straining at elevated temperatures affects the ductility of steel after it has cooled to the ambient temperature. Terazawa reported the results of applying various amounts of tensile strain at various temperatures to structural steel, as shown in Figure 5. (Ref. 5) Steel samples were strained (pre-strained) at different ratios of room temperature elongation. After the steel cooled, the retained ductility (retained elongation) was measured as a ratio of the original room temperature elongation. As Figure 5 indicates, for the same level of strain, the retained elongation after pre-straining at 1112°F is less than the retained elongation when the pre-straining was done at room temperature (control retained elongation).

Other laboratory investigations have shown a reduction in ductility of about 37 percent for No. 8 bars bent and straightened at 1400 to 1500°F around a former with a diameter 4 times the bar diameter (strain of about 20 percent), and a reduction in ductility of about 8 percent when the former diameter was 8 times the bar diameter (a strain of about 11 percent). (Ref. 6) The latter results generally support the trend shown in Figure 5. Therefore, for bend diameters of 6 and 8 bar diameters (WSDOT's recommended bend diameters for hot bending), reductions in ductility of approximately 25 and 10 percent, respectively, may be expected.

Another concern in hot bending is the effects of strain aging at elevated temperatures on steel embrittlement. It is not clear to what extent the results presented in Figure 5 and those in Reference 6 are influenced by aging at elevated temperatures. However, there are indications that when strained steel is kept at high temperatures (800 to 1300°F) for appreciable time periods (300 seconds), its ductile/brittle transition temperature can increase. (Ref. 5) On the other hand, for short time periods (30 seconds) the increase in the transition temperature is insignificant.

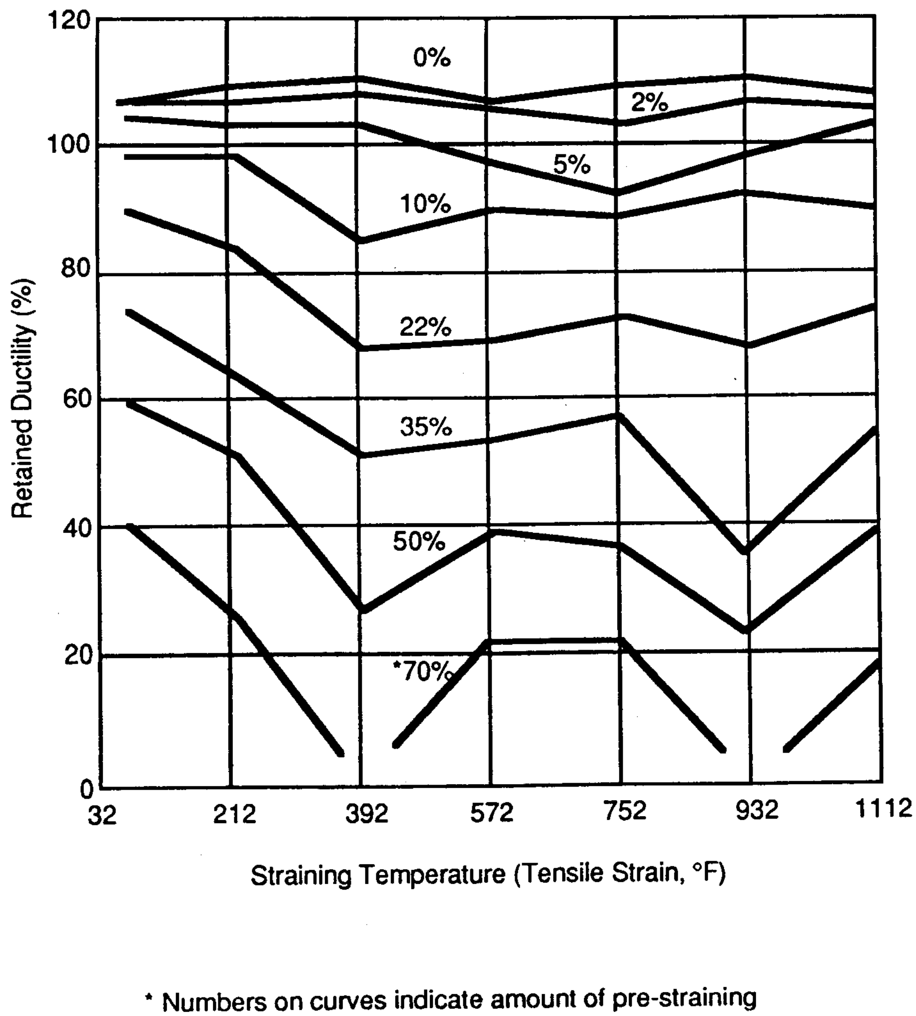


Figure 5. Room Temperature Tensile Ductility of Tensile Pre-Strained Specimens of 1-Inch Thick Killed Low-Carbon Steel (from Terazawa, ref. 5)

In hot-bending, aside from the effects of strain aging at high temperatures, reinforcing bars are subject to the effects of strain aging at room temperature after they have cooled to the ambient temperature. However, there are no test data to quantify these effects. If the effects of aging at elevated temperatures are ignored, the seismic concern regarding cold-bending/straightening bars is also valid for hot bending/straightening. This means that ASTM A615 grade 60 bars placed at locations designed to perform as seismic hinges can only be hot-bent and straightened at a former diameter of 30 bar diameters or more (depending on bar size), in order to avoid their brittle fracture in the event of a major seismic event.

SUGGESTED RESEARCH

Below is a suggested work plan for further research consisting of laboratory tests to verify and/or modify the findings of the current research.

TASK 1. EXPERIMENT DESIGN

Laboratory experiments are essential to the second phase of research. This task will design laboratory experiments to cold- and hot-bend bars, incorporating the variables listed below in the bending/straightening operation:

- bar diameter,
- bend diameter,
- bar orientation,
- number of bending cycles,
- mechanical properties before and after bending/straightening,
- heating temperature (hot-bending), and
- time of heating.

TASK 2. LABORATORY TESTS

Laboratory tests will be conducted in accordance to the experiments designed in Task 1. Bars bent and straightened under different conditions will be examined for the occurrence of cracking after the operation. If no cracking is present, bars will be tested to find their strength and ductility with and without application of strain aging.

TASK 3. DEVELOPMENT OF SPECIFICATION GUIDELINES

After the laboratory test data have been analyzed and interpreted, the results will be incorporated into the findings of the current research so that specification guidelines for bending/straightening reinforcing bars can be developed for WSDOT's consideration.

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VOLUME TWO

RESIN GROUTING EPOXY-COATED CONCRETE REINFORCING BARS

VOLUME TWO
TABLE OF CONTENTS

LIST OF TABLES

SUMMARY

Improving the conditions of existing bridges often requires the addition of new portions to the in-place structures. This may be done by drilling and grouting reinforcing bars to join the new portion to the existing structure. An example is bridge widening.

Recently, fast-curing resins have been developed that expedite the grouting of reinforcing bars. On the other hand, epoxy-coated bars are specified in certain bridge components to prevent bars' corrosion. Resin grouting of epoxy-coated bars has cast doubt on the bond of the resin grout to the epoxy coating.

The Washington State Department of Transportation (WSDOT) initiated this investigation to review its current procedure of resin grouting epoxy-coated bars and to recommend modifications to that procedure, if necessary, to ensure an adequate bond.

After information from relevant investigations was assimilated and analyzed, the results were compared to the Department's resin grouting procedure. Accordingly, recommendations were made for modification of the currently specified reinforcing bar embedment lengths and drilling and grouting procedures. Finally, a second phase of research consisting of laboratory tests was suggested to verify and/or modify the recommended procedures.

Presently, the embedment lengths required by WSDOT for resin grouting of epoxy-coated bars follow the embedment lengths required for cement grouting of uncoated bars. This study recommends a 50 percent increase in WSDOT's embedment lengths for resin grouting of anchor epoxy-coated bars, and a 30 percent to 40 percent reduction in WSDOT's embedment lengths for resin grouting of epoxy-coated stress transfer bars. Large drill hole diameters were found detrimental to the bond of resin grout to the adjacent concrete because of shrinkage of the grout. Also, core drilling can affect the bond adversely, unless the sides of the core are roughened. Although this study was specifically conducted to evaluate resin grouting of epoxy-coated bars, the findings can be made applicable to cement or resin grouting of uncoated bars.

FINDINGS

The following findings are based on the analyses and assessments conducted in this work of the data provided by previous laboratory investigations.

1. For uncoated bars, replacing cement grout with resin grout can reduce the minimum acceptable embedment length by at least 20 percent.
2. Replacing uncoated bars with epoxy-coated bars in resin grout can add up to 10 percent to the minimum acceptable embedment length.
3. Replacing uncoated bars with epoxy-coated bars adds about 15 percent to the minimum acceptable embedment length under cyclic loading.
4. With the minimum acceptable embedment length of an uncoated bar in cement grout as the reference value, the effect of resin grouting can be a 20 percent decrease. The effects of epoxy coating the bar and cyclic loading can be a 10 percent increase and a 15 percent increase, respectively. Thus, the combined net effect can be an increase of $0.80 \times 1.10 \times 1.15 \sim 2$ percent in the reference minimum acceptable embedment length.

RECOMMENDATIONS FOR TENTATIVE SPECIFICATION GUIDELINES FOR RESIN GROUTING EPOXY-COATED REINFORCING BARS

On the basis of the information assimilated and analyzed in this work, the following recommendations for drilling concrete and resin grouting epoxy-coated bars are made. These recommendations are intended to ensure adequate long-term bond strength while minimizing the cost of construction. These recommendations should be considered tentative, and they may be revised depending on the results of future laboratory tests.

1. Polyester resin grout is preferred to epoxy resin grout.
2. The minimum acceptable embedment lengths for anchoring and dowelling bars, or the basic embedment lengths where stress is transferred to the existing reinforcing bars, should be as follows. (Note that in the latter case the embedment length is equal to splice length, and it is determined from the basic embedment length by the AASHTO procedure.)

Bar Size	Minimum Embedment Length (inches)
4	10.5
5	11.5
6	12.5
7	15
8	18
9	22

The preceding embedment lengths should be reduced by 20 percent when there is at least a 3-in. edge clearance and a 6-in. bar spacing. For anchor bars, the embedment lengths may be reduced by the ratio of the actual embedded steel to the required steel. For dowel bars, where stress in the bar is insignificant, the currently specified WSDOT embedment lengths are preferred.

3. The drill hole diameter should be according to the resin manufacturer's recommendations. If there are no recommendations, the hole diameter

should be small enough to prevent the adverse effects of resin shrinkage on the bond with the concrete. In the absence of test data, the hole diameter may be 0.125 in. larger than the bar diameter.

4. Core drilling, if necessary, should be completed with a rotary hammer by roughening the sides of the core.

INTRODUCTION

Improving the conditions of deficient bridges is usually more economical than replacing them with new structures. Often this requires the addition of new portions to the in-place structure. An example is bridge widening. In certain types of reinforced concrete structures this type of improvement can be achieved by drilling into the existing concrete and grouting the reinforcing bars, rather than removing a portion of the existing concrete and exposing its reinforcement for splicing purpose. Drilling concrete and grouting reinforcing bars may have other applications in the construction of both existing and new bridges. Examples are anchoring concrete barriers, railings, and traffic signs to the structure. In the past, cement has been used extensively for grouting. However, this has changed recently with the development of fast-curing resins.

Grouting reinforcing bars with resins has caused two concerns where epoxy-coated bars are used. First, some authorities have cast doubt on the bond of the resin to the epoxy coating because of the smooth texture of the cured epoxy coating. Second, a chemical reaction may take place between the epoxy of the coated bar and the resin of the grout as a result of heat generated when the resin grout is curing. This reaction may adversely affect the bond. In either case, if the mechanical bond between the resin grout and epoxy-coated deformed bar does not provide the same strength as documented for uncoated deformed bars, then different embedment lengths need to be specified for various sizes of epoxy coated bars.

OBJECTIVES

The Washington State Department of Transportation (WSDOT) initiated this investigation with the following objectives:

- to review WSDOT's current design procedure for resin grouting epoxy-coated bars, and to compare that procedure with the relevant research findings; and

- to recommend modifications to WSDOT's current design procedure, if necessary, to ensure adequate bond

RESEARCH APPROACH

The first step in the research was to review WSDOT's design guidelines for specifying embedment lengths for resin grouted epoxy-coated reinforcing bars. Subsequent to this task, laboratory test data were collected from U.S. and foreign sources on the bond of cement grouted uncoated bars and the effects of resin grouting and epoxy coating of bars on that bond. After those effects were quantified, recommendations were made to modify the WSDOT specified embedment lengths for resin grouted epoxy-coated reinforcing bars. Finally, further research, consisting of laboratory tests, was suggested to verify and/or modify the findings of this work.

REVIEW OF WSDOT'S SPECIFICATION GUIDELINES

Presently, WSDOT specifies the same embedment lengths for grouting with cement and grouting with resin. Also, the specifications for embedding epoxy-coated bars are the same as those for uncoated bars.

WSDOT specifies that the embedment lengths given in Table 1 are the minimum acceptable for grouting No. 4, No. 5, and No. 6 bars into the existing structure. (Ref. 1) Concrete edge distance, bar spacing, concrete strength (if significantly different from 4000 psi), etc., must be considered before those values are used. WSDOT further specifies that drill holes should have a minimum clearance of 3 in. from the edge of the concrete. With a 3-in. clearance, the values in Table 1 should be reduced by 20 percent if the reinforcement is spaced at least 6 in. on the centers.

Generally, No. 6 and smaller bars are grouted as dowel bars, rather than having stress transferred to the existing reinforcing bars. Examples are transverse distribution steel in flat slab bridges and diaphragm steel in bridge widening. Dowel bars need a minimum embedment length to provide continuity. No. 6 and smaller bars may also be grouted as anchor bars. An example is the anchoring of traffic barriers to bridge decks. For both dowel and anchor bars, WSDOT specifies the embedment lengths in Table 1 as the minimum acceptable. However, for anchor bars, the embedment lengths in Table 1 are reduced by the ratio of the actual embedded steel to the required steel, based on the stress developed in the bar.

No. 7 and larger bars are generally grouted to transfer stress to the existing reinforcing bars (e.g., cross-beam main reinforcement in bridge widening). In this case, WSDOT specifies the embedment length to be at least equal to the minimum splice length determined by the AASHTO procedure. The AASHTO procedure determines the minimum splice length, from the basic development length, depending of the detailing of the bars. (Ref. 2) The basic development lengths for No. 11 bars and smaller are given in Table 2.

The information in Table 2 indicates that drilling into the existing structure for grouting No. 7 and larger bars requires holes deeper than 2 feet. WSDOT requires core drilling for these holes. (Ref. 1)

Table 1. WSDOT Required Embedment Lengths for Grouting Dowel and Anchor Bars into the Existing Structure (ref. 1)

Bar size	Drill hole for grout (in.)	Drill hole for resin (in.)	Embedded Length (in)	
			Grade 40	Grade 60
4	1-1/4	3/4	6	7
5	1-1/2	1	7	8
6	1-3/4	1	8	9

Table 2. WSDOT Required Basic Development Lengths for Stress Transfer Bars, for 4000 psi Concrete (ref. 2)

Bar size	Development Length* (in)	
	Grade 40	Grade 60
7	15	23
8	20	30
9	25	38
10	32	47
11	39	57

* Basic Development Length, $l_d = \dfrac{0.04\, A_b\, f_y}{\sqrt{f'_c}}$ but > $0.0004\, d_b f_y$

where

A_b = Area of bar, sq. in.

f_y = Yield strength of steel, psi

f'_c = Compressive strength of concrete, psi

d_b = Nominal diameter of bar, in.

INTERPRETATION OF INFORMATION, APPRAISAL, AND APPLICATION

The available literature does not give minimum embedment lengths for resin grouted epoxy-coated reinforcing bars. However, the literature is relatively rich regarding minimum embedment lengths for cement grouted uncoated bars. On the other hand, the results from a few laboratory tests give clues regarding the effects of resin grouting, as well as epoxy coating of bars, on the bond characteristics of uncoated bars in cement grout. Therefore, the information is assimilated in this chapter in three steps. First, the minimum acceptable embedment lengths between uncoated reinforcing bars and cement grout are determined. Second, those embedment lengths are modified by taking into account the effects of replacing cement grout with resin grout. Third, the latter embedment lengths are further modified in consideration of epoxy coating the uncoated bar.

EMBEDMENT LENGTHS FOR CEMENT GROUTED UNCOATED BARS

The New York State Department of Transportation conducted comprehensive pull-out tests to determine acceptable minimum embedment lengths for grouted uncoated bars. (Ref. 3) Grade 60 bars were placed in holes drilled in unreinforced concrete slabs and were grouted with a Type 2 cement grout. Consistent with the ultimate strength design method adopted by AASHTO (Ref. 2) to determine reinforcing bar development length, a minimum acceptance criterion equal to 125 percent of the yield strength of the bars was used. The minimum acceptable development lengths found in the New York experiment are shown in Table 3 for bars ranging in size from No. 5 to No. 9.

The New York experiment, based on tests conducted by the California Department of Transportation, limited the drill hole diameter for cement grouting to 0.25 in. larger than the nominal bar diameter (Table 3). This was done to keep the grout volume small, to minimize the shrinkage of the grout, and to improve the bond in the pull-out tests.

Table 3. Minimum Acceptable Embedment Lengths for Grade 60 Cement Grouted Uncoated Bars (adapted from ref. 3)

Bar size	Hole Diameter (in.)	Minimum acceptable embedment length* (in.)
5	7/8	9
6	1	10
7	9/8	12
8	1-1/4	14
9	1-3/8	17

* Criteria; 125% of bar yield strength or more; concrete strength > 4,000 psi

Table 4. Minimum Acceptable Embedment Lengths for Grade 60 Resin Grouted Uncoated Bars (adapted from ref. 3)

		Bar size	Hole Diameter (in.)	Minimum acceptable embedment length* (in.)
Polyester resin grout	Type A	5	3/4	5
		6	7/8	7
	Type B	5	3/4	6
		6	7/8	8

* Criteria; 125% of bar yield strength or more; concrete strength > 4,000 psi

THE EFFECTS OF RESIN GROUTING ON THE BOND

The New York experiment also included two types of polyester resin grout tested in conjunction with uncoated No. 5 and 6 bars. (Ref. 3) The minimum acceptable embedment lengths in this case, as shown in Table 4, are 40 percent to 20 percent less than those obtained for the cement grout, depending on the type of resin.

An investigation by the Army Corps of Engineers found that, in pull-out tests of No. 6 uncoated bars grouted into concrete by means of a polyester resin, an embedment length of 7.5 in. did not result in yielding of the steel. (Ref. 8) That embedment length is 25 percent less than the minimum acceptable embedment length for cement grouted No. 6 bars (Table 3), and it is considered unacceptable according to the AASHTO ultimate design criteria.

Therefore, in consideration of the bond characteristics of various types of resin, the minimum acceptable embedment lengths for resin grouted uncoated bars may be a conservative 20 percent less than those for cement grouted uncoated bars, the lower limit obtained in the New York experiment. (Ref. 3)

In the New York experiment (Ref. 3) for resin grouting, the drill hole diameter was 0.125 in. larger than the bar diameter. This measurement was based on the manufacturer's recommendations and was intended to minimize shrinkage, as discussed previously. In the Corps of Engineers' investigation, the hole diameter for resin grouting No. 6 bars was 0.375 in. larger than the bar diameter. This may be another reason for the lower bond strength in the latter investigation. The current WSDOT required drill hole diameter for No. 6 and smaller bars, as shown in Table 1, are larger than those used in the New York experiment. The difference, however, is mainly for cement grouting, rather than resin grouting. For No. 7 and larger bars, WSDOT may require core drilling, since the depth of embedment is 2 feet or more. Core drilling produces a smooth hole, which reduces the bond strength between the grout and adjacent concrete. To eliminate this problem, the sides of the core-drilled hole can be roughened with a rotary impact hammer. (Ref. 3) This provides a good mechanical interlock. (Ref. 4, 5)

THE EFFECTS OF EPOXY-COATING BARS ON THE BOND

A European study based on pull-out tests found that epoxy coating of bars did not decrease the uncoated bars' bond strength to a polyester concrete, but it did decrease the bond strength to an epoxy concrete by 10 percent. (Ref. 7) That research used the amount of slippage of the embedded bar as the criterion to determine the bond strength.

An implication of this finding is that, whatever the cause of the decrease in the bond strength (e.g., an inferior adhesion of the epoxy concrete to the cured epoxy coating, or the occurrence of an adverse chemical reaction between the two epoxies during the epoxy concrete's curing period), the bond strength of the resin grouted epoxy-coated bars may be assumed to be 10 percent less than that of the resin grouted uncoated bars. This decrease in bond strength is due to the unexpected performance of the resin grout when associated with the epoxy coating. A 10 percent decrease in the bond strength is equivalent to a 10 percent increase in the embedment length.

THE EFFECTS OF CYCLIC LOADING ON THE BOND

The effects of cyclic loading on the bond strength can be more significant for epoxy-coated bars than for uncoated bars. For epoxy-coated bars, the design of reinforced concrete bridge elements should consider those effects on the bond. Laboratory research conducted for the North Carolina Department of Transportation produced information regarding the fatigue bond characteristics of epoxy-coated bars. (Ref. 6) Epoxy-coated bars and uncoated bars were cast in cement concrete with varying embedment lengths and were subjected to numerous cycles of loading in a working stress range. After the completion of the cyclic loading, the specimens were unloaded and subjected to pull-out tests to determine the bond strength. The criterion for satisfactory bond strength was that the stress in the bar be at least 125 percent of the yield strength of the bar. On the basis of the results of these tests, the research recommended a 15 percent increase in development length for epoxy-coated bars.

THE COMBINED EFFECTS OF RESIN GROUTING, EPOXY COATING BARS, AND CYCLIC LOADING ON THE BOND

The New York experiment, which was based on static tests, recommended minimum acceptable embedment lengths for cement grouted uncoated bars, as shown in Table 3. (Ref. 3) With the embedment lengths in Table 3 considered to be the reference embedment lengths, the following discussion attempts to modify those reference embedment lengths for the combined effects of resin grouting, epoxy coating of bars, and cyclic loading on the bond.

The results of the New York experiment and the Corps of Engineers' investigations showed that replacing cement grout with resin grout for uncoated bars could reduce the minimum acceptable embedment length by 20 percent (a conservative figure). (Ref. 8) The European investigation indicated that replacing uncoated bars with epoxy-coated bars in resin concrete could add to the minimum acceptable embedment length by 10 percent. (Ref. 7) The North Carolina investigation recommended a 15 percent increase in the minimum acceptable embedment length when bars are epoxy coated in order to take into account the effects of cyclic loading. (Ref. 6) Therefore, the net effect of resin grouting, epoxy coating of bars, and cyclic loading on the reference embedment lengths given in Table 3 is an increase of 0.80 x 1.10 x 1.15 ~ 2 percent in those lengths. Table 5 gives the modified embedment lengths.

RECOMMENDED EMBEDMENT LENGTHS FOR RESIN GROUTING OF EPOXY-COATED BARS

The embedment lengths in Table 5 are based on an at least 3-in. edge clearance and 6-in. bar spacing. (Ref. 3) Thus, for a general case, those lengths should be divided by 0.8. (Ref. 2) Table 6, which is based on the latter modification, suggests minimum acceptable embedment lengths for resin grouted epoxy-coated bars.

Table 5. Suggested Embedment Lengths for Grade 60 Resin Grouted Epoxy-coated Bars Subject to Cyclic Loading (special case)*

Bar size	Minimum acceptable embedment length** (in.)
5	9
6	10
7	12
8	14.5
9	17.5

* For edge clearance of at least 3 inches and bar spacing of at least 6 inches

** Criteria; 125% of bar yield strength or more

Table 6. Suggested Embedment Lengths for Grade 60 Resin Grouted Epoxy-coated Bars Subject to Cyclic Loading (general case)*

Bar size	Minimum acceptable embedment length (in.)
5	11.5
6	12.5
7	15
8	18
9	22

* No consideration for edge clearance and bar spacing

A comparison of the embedment lengths in Table 6 with WSDOT's required embedment lengths (Table 1 and Table 2) suggests that a 50 percent increase in WSDOT's embedment lengths is necessary for resin grouting of epoxy-coated dowel bars and anchor bars (generally No. 6 and smaller bars). Also, that comparison suggests that a 30 percent to 40 percent reduction in WSDOT's embedment lengths is necessary for resin grouting of stress transfer bars (generally No. 7 and larger bars). The latter suggestion is justified, since WSDOT's embedment lengths for stress transfer bars are based on AASHTO's basic development lengths, which were developed conservatively for new constructions where the bars can be inexpensively spliced. (Ref. 2) The suggested embedment lengths in this work for stress transfer bars can result in considerable cost saving in drilling concrete and grouting of those bars.

SUGGESTED RESEARCH

Below are suggestions for further research, consisting of laboratory tests, to verify and/or modify the recommendations of the current investigation regarding resin grouting of epoxy-coated bars.

TASK 1. EXPERIMENT DESIGN

In this task, pull-out laboratory experiments will be designed to determine the minimum acceptable embedment lengths for epoxy-coated bars of different sizes. Consistent with the AASHTO requirement, the embedment length should be large enough to produce at least 125 percent of the yield strength in the bar. In addition to bar size, the variables in the experiment will include type of resin grout (i.e., epoxy versus polyester) and hole diameter. Control specimens will be made with uncoated bars.

Also, flexural members extended with resin grouted epoxy-coated bars will be designed. This experiment will evaluate the recommended embedment lengths when transfer of stress to the existing reinforcing bars is a concern. Ultimate moment capacities will be obtained and compared to monolithically cast control beams' capacities.

TASK 2. LABORATORY TESTS

This task will include laboratory experiments conducted in accordance with the designs produced in Task 1. The University of Washington Civil Engineering Structural and Material Laboratories are available for the laboratory tests.

TASK 3. DEVELOPMENT OF STANDARD SPECIFICATION

On the basis of the analysis and interpretation of the test data and the background data available from the current investigation, specification guidelines for resin grouting of epoxy-coated bars will be developed for WSDOT's consideration.

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