INVESTIGATING THE SEISMIC BEHAVIOR OF HIGH STRENGTH STEEL RC COLUMNS

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Abstract

The advantages of high strength steel reinforcement for use in capacity protected members and actions are self-evident as it reduces congestion and cost. However, in order to extend the use of high strength steel to members forming plastic hinges, the material must possess sufficient inelastic strain capacity and ductility. It is well established that the uniform elongation of Grade 60 steel is at least 33% higher than that of Grade 80 steel. Such a difference in uniform elongation has been suggested to reduce member deformation capacity. An extensive program at NC State was developed to quantify seismic design parameters of RC bridge columns using Grade 80 reinforcing steel. The study focuses on evaluating both the performance and required capacity of bridge columns through experimental testing to evaluate the rebar behavior along with access the required seismic strain demand. From large scale column tests, early bar fracture after bar buckling was noted and a new material test was developed, referred to as the 'Buckled Bar Tension Test' (BBT). The test aims to assess the impact of buckling induced bar curvature (measured as a bending strain along the bar cross-section) on axial tension fracture strain capacity. In this paper, the results of BBT tests conducted on both Grade 60 and Grade 80 steel are compared against the results of large scale reinforced concrete column tests. Results showed a correlation between the two, which was then used as a guide for the selection of reinforcing steel for subsequent column tests. Among the variables that have been shown to impact BBT test results are rebar height to rib radius, as well as rebar manufacturing process. Rebar geometry was assessed with 3D laser scanning with the resulting mesh used to develop computational finite element models to correlate to BBT test results. The models were able to match the horizontal displacements of the buckled bar of the BBT tests. Beyond the experimental results, the finite element analysis shows local bending strains and concentrated areas of high strain. Lastly, a column demand study is presented to evaluate the required capacity of RC bridge columns by defining the strain demand through nonlinear time history analysis. The objectives of this study are to evaluate the strength and design of columns to demonstrate Grade 80 steel has adequate displacement capacity for the expected seismic demand.

Keywords: grade 80 steel; column; concrete; buckling

1. Introduction

In the seismic design of reinforced concrete (RC) members, Grade 60 reinforcing steel has been universally utilized in members forming plastic hinges. The advantages of high strength reinforcing steel can optimize designs by reducing material cost, construction time, and congestion. Improvements also arise from minimized reinforced concrete sections and improved confinement to resist bar buckling. The use of high strength reinforcing steel, such as Grade 80 steel, remains restricted in many design codes due to the lack of sufficient research necessary to fully understand and quantify its behavior under cyclic loading [1]. In line with capacity design principles, the damage of seismic members occurs in locations of inelastic deformation, known as plastic hinges, which act to dissipate energy [2]. Through mindful detailing for the anticipated seismic demands, these locations are designed to fail in flexure by allowing plastic rotation of the connection. The typical failure mode of RC columns is the fracture of previously buckled longitudinal reinforcing bars, which causes significant loss of capacity of the structural system.

Although the advantages of Grade 80 steel are self-evident, structures with this reinforcement must remain ductile to ensure that plastic hinges are able to form and other members are to remain capacity-protected. The reinforcing steel must be proven with confidence to have enough inelastic strain capacity and adequate strain hardening. To ensure that this performance is achieved, many concepts need to be understood about the behavior of Grade 80 steel. To understand the seismic behavior of high strength reinforcing steel, an extensive experimental program at North Carolina State University (NCSU) was created with the main objective to quantify seismic design parameters of RC bridge columns using Grade 80 reinforcing steel [1]. This requires that the performance of the reinforcing steel under the expected seismic demands be understood on a material, member, and global level. The aim of this study is to quantify design parameters such as plastic hinge lengths and spread of plasticity, strain limits, hysteretic energy dissipation, and required strain demand.

Steps in evaluating the seismic behavior of Grade 80 reinforcing steel are focused on addressing both the performance and the required capacity of the columns in parallel. This research investigation was divided into an experimental study and a column demand study. The experimental approach seeks to determine the inherent structural capacity and seismic performance of Grade 80 steel through large-scale column tests and material tests to evaluate rebar behavior. The column demand study attempts to evaluate the actual strain demand required of these columns in order to demonstrate that Grade 80 steel has adequate displacement capacity for the expected seismic load.

2. Background

Implementation of Grade 80 steel in design requires a comprehensive understanding of how the reinforcing steel will behave on a material level. This extensive study of Grade 80 steel began with Overby et al. [3] which led to recommendations of material properties of A706 Grade 80 steel. To characterize the stress-strain parameters, approximately 800 tensile tests were performed on ASTM A706 Grade 80 reinforcing bars ranging from #4 to #18 sizes. These bars were all manufactured using a micro-alloying process and they corresponded to three different mills with three heats for each bar size. This work greatly contributed to extending the database of stress-strain behavior of A706 Grade 80 bars by over 650%. Previously, the publicly available test data only consisted of twelve tensile tests [1].

This investigation was the first step in the experimental study to understand the inelastic behavior of Grade 80 reinforcing steel that is necessary to use it in members forming plastic hinges. The key results were statistical quantification of the uniaxial stress-strain behavior of Grade 80 steel that can be utilized for future design parameters and material modeling. Another significant observed outcome was that Grade 80 bars had reduced axial strain at maximum stress and lower ultimate tensile strength to yield strength (T/Y ratio) compared to Grade 60 reinforcing steel. The investigation statistically represented the material properties of yield strength, yield strain, the onset of strain hardening, tensile strength, and tensile strength at max stress

[3]. Many of these recommended design parameters were included in the 2018 version of the Caltrans Seismic Design Guidelines.

Outside of this study conducted at NCSU, there have been limited column tests conducted with A706 Grade 80 steel. For example, circular columns studying both yield strength and aspect ratio by Barbosa et al. [4] found that A706 Grade 60 and Grade 80 columns had comparable displacement capacities. Reinforcing this finding, Sokoli and Ghannoum [5] conducted reverse cyclic tests on square columns reinforced with A706 Grade 80 steel with high axial loads and found similar results. Potential parameters influencing the seismic behavior of Grade 80 reinforcing bars are rib radius and manufacturing process. Restrepo-Posada [6] suggested early fracture of bars may be a result of cracks that form at the base of the rebar rib while loaded in compression. These cracks cause stress concentrations that are likely to result in fracture on the subsequent tension cycle. Rocha et al. [7] investigated locations of high stress concentrations on bars manufactured with a quench-and-self-tempering (QST) process. Finite element analyses showed that stress concentrations were influenced by the rib radius to height (r/h) ratio with high stresses at the rib to shaft interface. In an investigation of column demand, a seismic demand study was conducted by Huang et al. [8] analyzing 60 bridge configurations with single column bents traditionally representative of California practice. Seismic demand was found from finite element models of these bridges, which were evaluated using nonlinear time history analysis under multiple characteristic ground motions. All of these past investigations provide a basis to better understand the performance of Grade 80 steel and help to establish methods and equations for design.

3. Reinforced Concrete Column Tests

With an enhanced understanding of the material properties of Grade 80 steel, the next step of the research study was to investigate the expected seismic behavior and suitability of using high strength reinforcing steel for structural members. The goal of this study is to expand the limited experimental database of Grade 80 steel column tests to improve the understanding of its inelastic behavior under seismic loading. This investigation has been divided into phases consisting of large-scale experimental testing of circular concrete bridge columns reinforced with Grade 80 longitudinal and transverse steel.

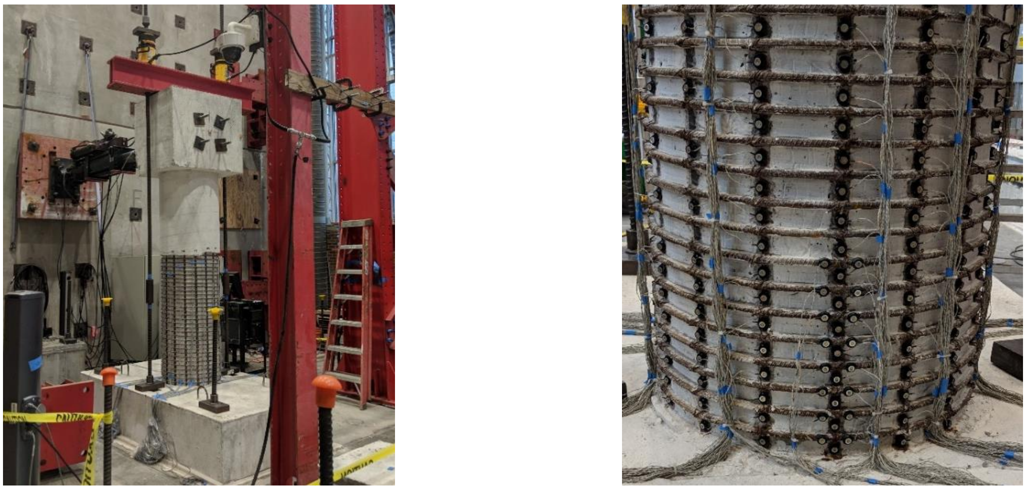
All of these tests were subjected to three-cycle reverse cyclic loading calculated from displacement ductility. Additionally, each test specimen was compared with columns constructed with Grade 60 reinforcing steel with matched strength or detailing imposed under similar loading and conditions. The test matrix outlining the detailing for past and present columns in this study is shown in Table 1. Currently, 8 large-scale columns reinforced with A706 Grade 80 longitudinal and transverse steel have been tested. The manufacturing process varied per mill with Mill 1 and 2 using a micro-alloying process and Mill 3 using a quench-and-self-tempering process.

An optical measuring system, Optotrak Certus HD, manufactured by Northern Digital, Inc. [9] was used to capture the three–dimensional position of the longitudinal bars and spirals in the plastic hinge throughout the test. For instrumentation, the concrete cover was blocked out for the bottom half of the columns exposing the reinforcing steel as can be seen in Fig. 1. The resulting data was used to calculate the characteristic properties of the column tests such as strain and buckled shapes of the longitudinal bars.

The first large-scale experimental tests in “Phase 1” were conducted by Barclay and Kowalsky [10] testing four reverse cyclic columns tests reinforced with ASTM A706 Grade 80 steel manufactured with a micro-alloying process (Mill 1). In comparison to ASTM A706 Grade 60 column tests matching detailing and strength capacity, it was found that the Grade 80 columns had earlier fracture at lower displacement capacity after bar buckling had occurred.

Table 1 – Grade 80 Column Tests

	Column Number	Mill	Dia.	L/D	Longitudinal Steel	Transverse Steel (Spirals)	Axial Load Ratio
Phase 1	1	1	24”	4	16 #6 (1.6%)	#3 @ 2.0” (1.0%)	5%
	2	1	24”	4	16 #6 (1.6%)	#3 @ 2.0” (1.0%)	10%
	3	1	24”	4	16 #6 (1.6%)	#3 @ 2.75” (0.7%)	5%
	4	1	24”	4	16 #6 (1.6%)	#3 @ 1.5” (1.3%)	5%
Phase 2a	1	3	24”	4	16 #6 (1.6%)	#3 @ 1.5” (1.3%)	5%
	2	2	24”	4	16 #6 (1.6%)	#3 @ 1.5” (1.3%)	5%
	3	3	24”	4	16 #6 (1.6%)	#3 @ 2.0” (1.0%)	10%
	4	2	24”	4	16 #6 (1.6%)	#3 @ 2.0” (1.0%)	10%
Phase 2b	5	3	18”	5.33	10 #6 (1.7%)	#3 @ 2.0” (1.3%)	10%
	6	2	18”	5.33	10 #6 (1.7%)	#3 @ 2.0” (1.3%)	10%
	7	3	18”	5.33	10 #6 (1.7%)	#3 @ 2.0” (1.3%)	20%
	8	2	18”	5.33	10 #6 (1.7%)	#3 @ 2.0” (1.3%)	20%
	9	3	18”	8.67	10 #6 (1.7%)	#3 @ 2.0” (1.3%)	10%
	10	2	18”	8.67	10 #6 (1.7%)	#3 @ 2.0” (1.3%)	10%



(a) (b)

Fig. 1 – (a) Column Test Setup [1], (b) Optical Instrumentation of Reinforcing Steel

4. The Buckled Bar Tension Test

To investigate the causes of the early bar fracture in Phase 1, a material test was created at NCSU designed to mimic this post-buckling behavior. This material test was called the buckled bar tension (BBT) test and was designed to replicate the reverse tension-compression loading on a longitudinal reinforcing bar during a cyclic column test. From this study, it was found that the buckled reinforcing bar would reach a “critical bending strain" at the transition of ductile and brittle failure. Correlation between experimental results found that when the bar surpassed this critical bending strain (ε_{cr}) in compression, it would likely result in a brittle fracture upon reverse tensile loading. This parameter could be effective in estimating the expected performance of the reinforcing steel under cyclic loading. This ε_{cr} was hypothesized to be linked to differences in displacement capacities of the Grade 80 versus Grade 60 columns in Phase 1 [11].

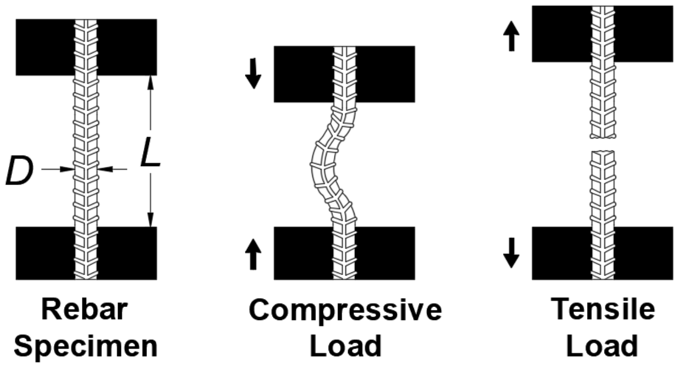


Fig. 2 – Loading Sequence of Buckled Bar Tension Test [11]

The BBT test can be used to understand the strain distribution of a buckled bar. The bar specimen is loaded in compression to a selected buckled curvature and then pulled in tension until fracture as shown in Fig. 2. The position of the bar throughout the entire test is measured through a 3D optical measuring system of LEDs. The resulting rebar displaced shapes were differentiated twice to calculate buckling induced curvature on the bar cross-section.

Various reinforcing steel specimens were tested with the BBT test from different strengths of steel (Grade 60, 80, 100), mills, bar size, and manufacturing process. Samples of reinforcing steel bars for each mill were subjected to increased curvature and the tensile failure modes were classified as ductile or brittle fractures. Typical characteristics of ductile failures result in straightening of the rebar and necking at fracture while a brittle failure results in little to no elongation of the bar and a flat fracture surface. These two different failure modes from BBT tests can be seen in Fig. 3.

(a) (b)

Fig. 3 – Rebar Failure Modes, (a) Ductile, (b) Brittle [11]

The two methods investigated for producing high strength steel include the traditional micro-alloying process and quench-and-self-tempered process. The bending strain from these tests serves as an indicator of the curvature that may cause undesirable results. In processing these results, the bending strains at the center of the bar are compared to the axial strain at maximum stress as shown in Fig. 4 (a) – (d). The drop-off of the axial strain at maximum stress captures the ductile-brittle transition of the steel which can be characterized by a critical bending strain range.

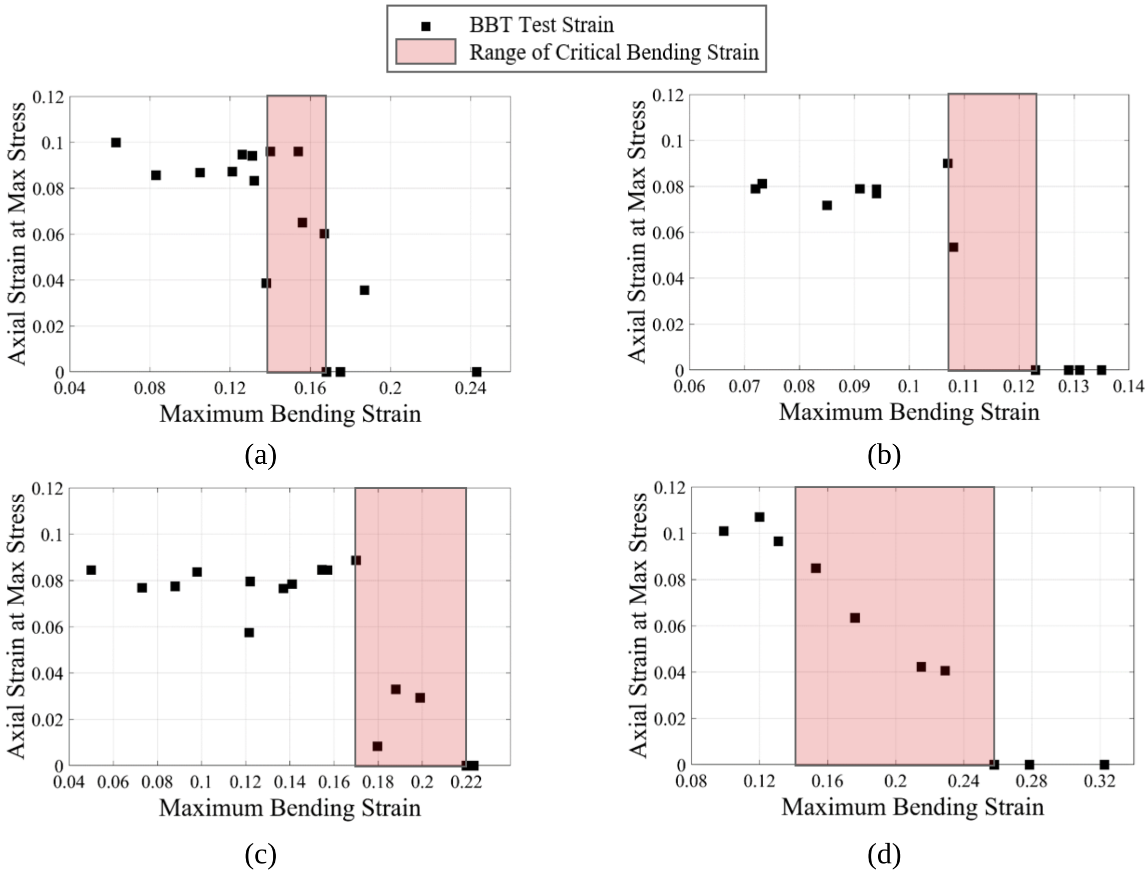


Fig. 4 – BBT Tests, (a) Mill 2, Grade 60, (b) Mill 1, Grade 80, (c) Mill 2, Grade 80, (d) Mill 3, Grade 80

From the BBT tests, it was found that the key parameters that largely influenced ε_{cr} were the rib radius of the bars and the manufacturing process. Conclusions from these tests linked larger rib radius to greater critical bending strains. Additionally, it was found that steel manufactured through the QST process resulted in improved bending strain values [11]. The BBT tests have been used as a baseline for proceeding column tests to indicate which reinforcing steel to test in large-scale column tests.

5. Key Parameters Influencing Critical Bending Strain

5.1 Manufacturing Process

The two manufacturing processes to produce Grade 80 reinforcing steel investigated in this study were micro-alloying and quench-and-self-tempered methods. The traditional micro-alloying process involves the adjustment of alloys to tailor the chemical composition to achieve higher strength steel. Conversely, QST steel gains its strength through rapidly cooling the rebar after it is rolled. This quenching technique creates a tough martensitic exterior tempered from the hot ductile ferrite-perlite core. This method is a simpler manufacturing process that could allow for rapid production and cost savings by requiring less carbon and expensive alloys.

This difference in manufacturing processing causes QST to typically have a lower T/Y ratio and significantly longer yield plateau compared to the micro-alloyed bars. This lower T/Y ratio can be a drawback for using QST processing since it often cannot be classified as ASTM A706 due to the specified limit of the T/Y ratio of 1.25 [12]. In the seismic design of RC members, the T/Y ratio takes on an important role in the characterization of the spread of plasticity. Currently, there seems to be a lack of research to back an acceptable T/Y ratio required to provide an adequate spread of plasticity and rotation in the plastic hinge.

In this experimental program, the implications of using a lower T/Y ratio have been investigated through column test results of Phase 2a, material tests, and an analytical parametric study.

From these results, significant outcomes resulting from using a lower T/Y ratio are lower cyclic hardening, lower displacement capacity, and concentrated plasticity [1]. Extension of this analytical study to fiber models will be conducted to investigate the impact of using different T/Y ratios. There are clear differences in material behavior that need to be studied to characterize material modeling. Additionally, large-scale column tests will be tested to further explore the seismic behavior of columns reinforced with QST steel.

5.2 Rebar Rib Geometry

Considering the performance of reinforcing steel, a clear understanding of the impact of rib geometry was required to understand its seismic behavior. Due to the difficulty of accurately measuring rebar radius, a more quantitative method was developed to evaluate the impact of the reinforcing rib geometry. Reinforcing steel was sampled from the #6 longitudinal steel bars from the three mills evaluated in the Grade 80 column tests in Phase 1 and 2a. These rebar specimens were cut into 3 inch lengths and 3D laser scanned using a NextEngine Laser Scanner [13]. The scans were converted into solid elements and measurements were able to be obtained using 3D solid modeling software. Multiple scans were collected for each mill and average geometric measurements for rib radius, rib height, and radius/height ratio (r/h) are shown in Table 2. Differences in the rebar geometries can be seen in Fig. 5 (b) – (d).

Table 2 – Geometric Parameters of Grade 80 Reinforcing Steel

Mill	Manufacturing Process	Rib Radius, in [mm]	Rib Height, in [mm]	r/h
1	MA	0.109 [2.79]	0.0609 [1.55]	1.81
2	MA	0.167 [4.24]	0.0545 [1.38]	3.07
3	QST	0.135 [3.44]	0.0414 [1.05]	3.27

The rib geometry pattern of the scans was extruded to the typical specimen size of 17 inches which equates to a 7 inch clear length and length over diameter, L/D, of 9.3. The reinforcing steel models were meshed into solid quadratic tetrahedral elements with curved boundaries and midside nodes as seen in Fig. 5 (a). The average number of elements for all of the specimens was 190,000 elements. The specimens were exported to the finite element analysis (FEA) software, ANSYS, and static structural analyses were conducted.

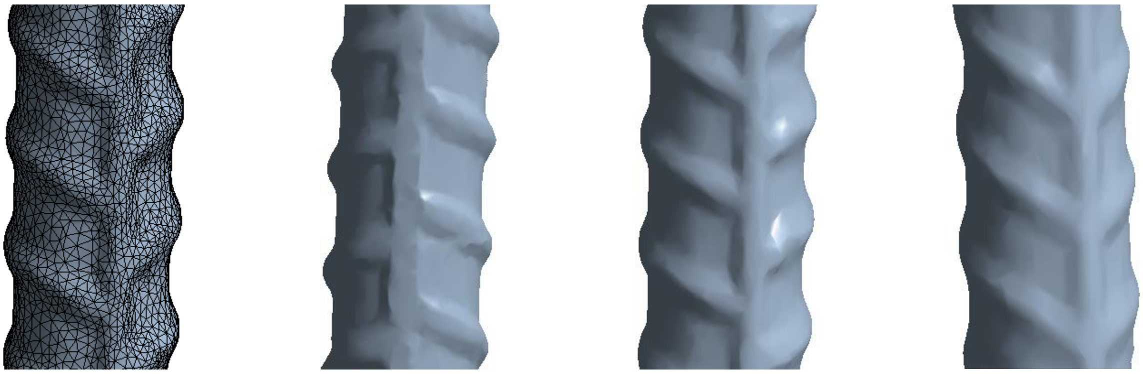
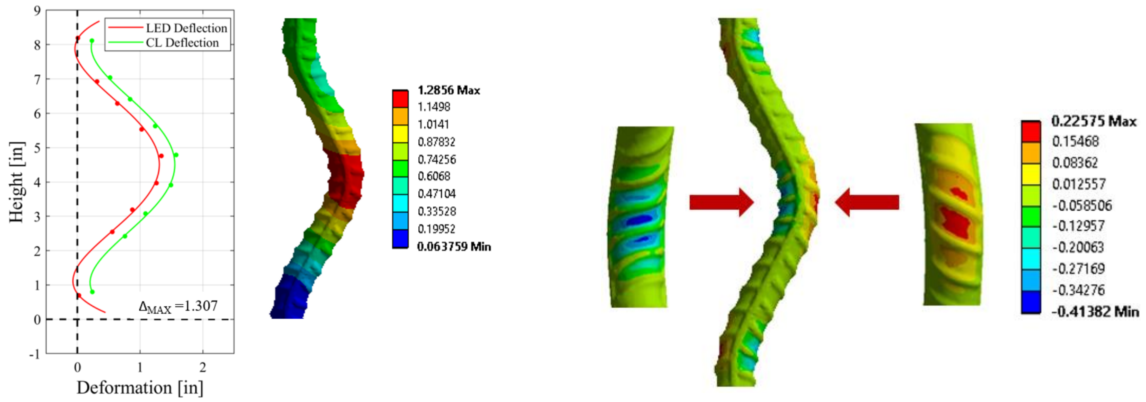

(a) (b) (c) (d)

Fig. 5 – BBT Tests, (a) Mill 2, Grade 60, (b) Mill 1, Grade 80, (c) Mill 2, Grade 80, (d) Mill 3, Grade 80

Finite element models with fixed boundary conditions replicating the BBT tests as 5 inch grips were applied to the top and bottom of the bar. The exact displacement histories of the BBT was applied to the FEA model. Additionally, a small lateral force was applied to the center of the bar to force the mode of buckling.

A plasticity material model was applied with multilinear kinematic hardening using the average stress-strain data from material tests for each mill.



(a) (b) (c)

Fig. 6 – Out-of-Plane Displacement: (a) BBT, (b) FEA BBT; Bending Strain: (c) FEA BBT

Replicating the same loading history of the BBT tests, the exact displacement of the test was applied to the ANSYS model. A total of 32 tests, eight from each Grade 80 mill were conducted to mimic the experimental BBT tests. BBT loading histories were chosen around the ductile-brittle transition region. Similar to the method that the critical bending strain range was found for the experimental BBT tests, the local critical bending strain range was established for each mill and can be seen in Fig. 8. This value is essential to compare the test parameters of the FEA model and evaluate possible brittle failures.

Table 3 – Critical Bending Strain of BBT Tests and FEA BBT

Mill	Manufacturing Process	Critical Bending Strain, ε_{cr}	
		BBT	FEA BBT
1	MA	0.11-0.12	0.27-0.3
2	MA	0.17-0.22	0.3-0.34
3	QST	0.14-0.26	0.27-0.4

The measured strain from the experimental BBT test differed from the strain results of the FEA analysis as can been seen in Table 3. The BBT test data utilizes the Optotrak measuring system to obtain the position of the bar throughout the test to calculate the strains. The resulting critical bending strain is computed at the center of the bar as shown as ε_a in Fig. 7. The FEA model evaluates the local strain at each solid element under the same prescribed displacement history as the BBT tests. The entire strain field of the reinforcing bar can be evaluated showing areas of high strain concentrated between the rebar ribs as seen in Fig. 6 (c). The local critical bending strain is found on the compressive face of the reinforcing steel bar at the maximum lateral deformation shown as ε_c in Fig. 7. At the most buckled state of the bar, the maximum lateral displacement from the experiment and the FEA model matched very closely for all tests providing experimental validation as shown in Fig. 6 (a) and (b).

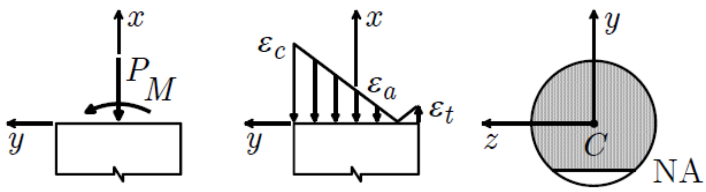


Fig. 7 – Strain State of Buckled Bar

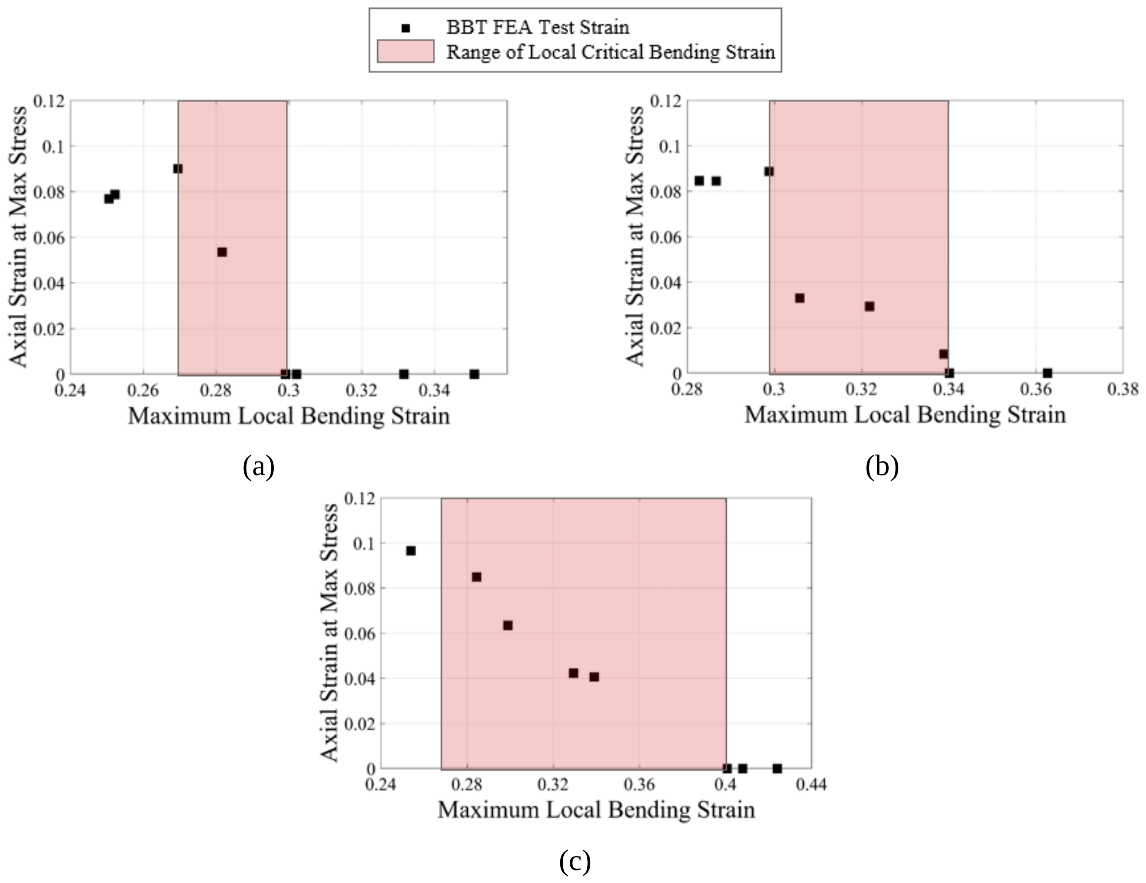


Fig. 8 – FEA BBT Test Results, (a) Mill 1, Grade 80, (b) Mill 2, Grade 80, (c) Mill 3, Grade 80

6. Bending Strain in RC Column Tests

To study the key findings from Phase 1 and continue to increase the experimental database of Grade 80 columns, specimens were constructed to test the hypothesis that reinforcing steel resulting in high critical bending strain would correlate to improved displacement capacity [11]. As a result, in Phase 2a conducted by Manhard [1], two columns were constructed with the larger rib radius steel (Mill 2) and the other two used QST steel (Mill 3) which matched the exact detailing of Phase 1 columns. Results from these tests show favorable lateral displacement under buckling compared to Phase 1 and matched displacement capacity to A706 Grade 60 columns.

A comparison of bending strain histories for the fractured bars of the three Grade 80 steel mills and the Grade 60 steel is shown in Fig. 9. A summary of the critical bending strain ranges can be found in Table 4. The data set includes eight Grade 80 [1, 10] columns and 16 Grade 60 columns [14]. The range of column detailing included longitudinal steel ratio (1.6 to 3.1%), transverse steel ratio (0.5 to 1.3%), axial load ratio (5 to 20%), and aspect ratio (4 to 8.67). The range of critical bending strain displays the lower bound of the lowest strain before fracture and upper bound of the highest strain before fracture.

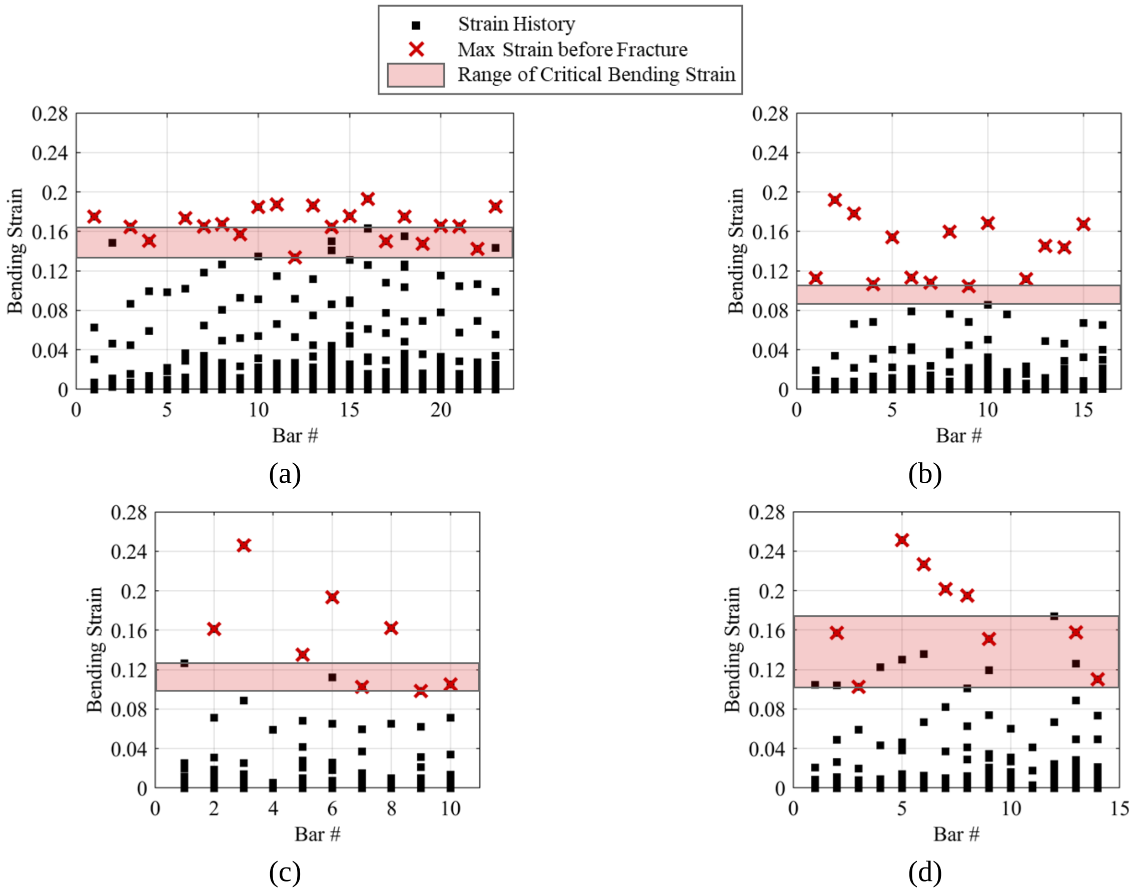


Fig. 9 – Bending Strain Histories, (a) Mill 2, Grade 60, (b) Mill 1, Grade 80, (c) Mill 2, Grade 80, (d) Mill 3, Grade 80 [1]

Table 4 – Critical Bending Strain of BBT Tests and FEA BBT

Mill	Designation	Manufacturing Process	Critical Bending Strain, ε_{cr}	
			BBT	Column
1	M1-80	MA	0.11-0.12	0.09-0.10
2	M2-60	MA	0.14-0.17	0.13-0.16
	M2-80		0.17-0.22	0.1-0.12
3	M3-80	QST	0.14-0.26	0.1-0.17

Comparatively, the bending strain histories showed that QST columns had a large critical bending strain range in contrast to other column tests. This is similar to the material tests seen in Fig. 4 (d), which shows the decreased axial strain is progressive, resulting in a large critical bending strain range. Mill 1 still has the lowest range of bending strain which validates a possible relationship between the seismic performance and both the manufacturing process and rib geometry.

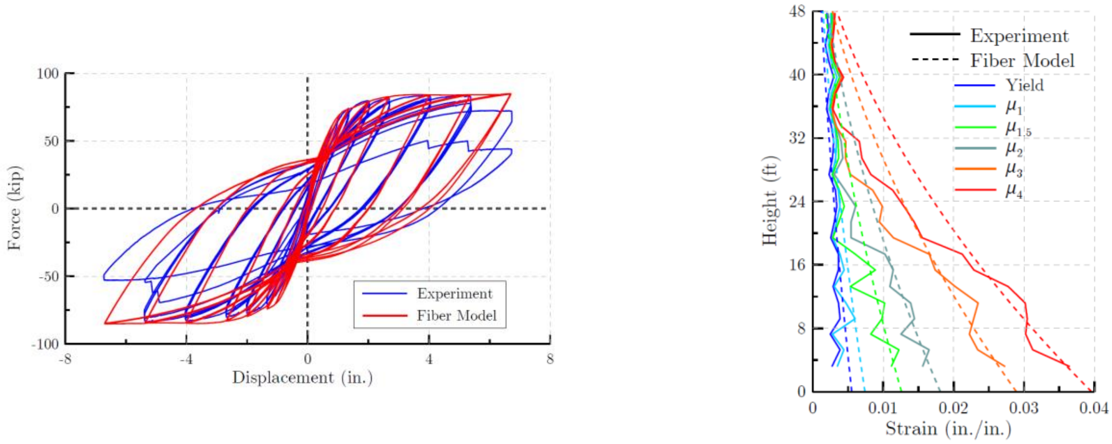
The Phase 2a column tests showed larger displacement capacity at fracture compared to Phase 1 columns and similar displacement and performance of the Grade 60 comparison columns. However contradicting the initial theory, the columns did not significantly improve in the bending strain capacity prior to fracture as the material tests suggested. The critical bending strain values were not as large as the Grade 60 comparison column tests. This discrepancy between the tests may be correlated to an inaccurate method for calculating bending strains for the column longitudinal bars. This process could improve by a better definition of the mathematical bar shape functions [1].

7. Column Demand Study

The column demand study seeks to determine the required capacity of columns reinforced with Grade 80 steel in terms of strain demand. This study evaluates the strain demand using two methods of equivalent strength and displacement analysis. The equivalent strength analysis focuses on columns detailed with matching capacity using Grade 60 and Grade 80 steel while the equivalent displacement analysis evaluates differences in columns designed for both grades of steel. Regardless of the capacity demonstrated in the experimental phase, the performance may be acceptable based on the actual demand required. This may be influential since the use of higher-strength steel can be substantially beneficial for high seismic areas.

The first step of the equivalent strength analysis was to identify several prototype bridges that are representative of a high seismic region. This study focuses specifically on typical column design variables of California practice. The varying parameters include column fixity, column diameter, height, axial load ratio, longitudinal steel ratio, and transverse steel ratio. Columns with matched strength were accessed accounting for identical transverse steel detailing for confinement and consistent longitudinal bar size for designs. For this study, 586 column designs were established varying the typical design parameters. To comprehend the implications of using higher strength steel, the strain demand of these designs will be compared for various earthquake intensities. In the equivalent displacement analysis, a study of the design for various strengths of steel was investigated. Utilizing displacement-based design, columns of various detailing were designed for the same target displacements based on the drift. Geographically, the design was based on several locations in California. Differences in design and resulting strain demands will be compared for Grade 60 and Grade 80 steel.

The final results of both methods will be the strain demand for reinforced concrete columns with Grade 60 and Grade 80 steel. These column designs have been modeled as fiber models in OpenSees and have been experimentally calibrated to column and material tests. The force-displacement and strain profiles from the fiber model show good agreement up to bar buckling in comparison to 12 column tests of various detailing, mills, and grades of steel as seen in Fig. 10. The next steps of this study are to conduct various nonlinear time history analyses that will result in strain demand in column plastic hinges. The outcome will explore the implications of design parameters and design results of various grades of reinforcing steel.



(a) (b)

Fig. 10 – Comparison Results from Phase 2a, Column 4 (a) Force-Displacement, (b) Strain Profile

8. Conclusions and Future Work

This comprehensive evaluation seeks to expand the study of high strength reinforcing steel to gain broader knowledge that can contribute to using Grade 80 steel for seismic design. To accomplish this, many different methods must be taken to fully understand the spectrum of components influential to its behavior under seismic loads.

Significant parameters that have been noted to influence the seismic behavior of Grade 80 steel include rebar geometry and manufacturing process. Both parameters will continue to be explored to evaluate their influence and study specific criteria that may improve their performance. On the experimental front, the last portion of this phase of columns tests will seek to not only match but exceed strength through unique detailing. Additionally, the possibility of shake table tests may be implemented to gain a more realistic behavior of the columns versus the current 3 cycle loading set that is being conducted. This opportunity to further explore the behavior of this material both experimentally and computationally will allow for the quantification of the seismic performance of Grade 80 rebar.

9. Acknowledgements

The research outlined in this paper was supported by the California Department of Transportation (Caltrans). Their support is gratefully appreciated, and it should also be noted that the results discussed here are those of the authors alone and should not be construed as to imply endorsement by Caltrans. The authors would also like to acknowledge the material donation from the reinforcing steel mills (Nucor, Gerdau, and CMC). Additionally, the authors wish to thank the technical staff and undergraduate assistants of the Constructed Facilities Laboratory at North Carolina State University, without whom this work would not have been possible.

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