ATC-98 Project on Seismic Design of Concrete Structures with High-Strength Reinforcement

Dominic J. Kelly[1], Andres Lepage[2], David Mar[3], Jose I. Restrepo[4], Joseph C. Sanders[5], and Andrew W. Taylor[6]

Synopsis: In the U.S., the specified yield strength of primary reinforcement of special concrete moment frames and special concrete shear walls may not exceed 60 ksi. As part of the Applied Technology Council ATC-98 Project, the use of high-strength reinforcement as primary reinforcement for seismic-force-resisting systems (SFRS) was studied. The purpose of the project was to examine the feasibility of using reinforcement with a specified yield strength greater than 60 ksi and, if feasible, to recommend changes in design requirements and reinforcing steel specifications. The use of ASTM A706 Grade 80 was specifically examined, and it was determined that Grade 80 reinforcement could be used with little change to ACI 318. It was also found that additional research is needed before complete recommendations for use of Grades 100 and 120 reinforcement can be made.

Keywords: Seismic, design, concrete, high-strength, reinforcement

INTRODUCTION

Reinforced concrete buildings designed and constructed in areas with high seismic hazard in the U.S. generally use American Society for Testing and Materials (ASTM) A706 Grade 60 flexural reinforcement in members resisting earthquake demands. However, reinforcement producers in the U.S. are now capable of manufacturing ASTM A706 Grade 80 reinforcement, which was introduced into the ASTM standard during the 09a revision, and are working towards producing even higher grades of reinforcement with ductile properties. The Applied Technology Council (ATC) formed a team of practitioners and researchers for ATC Project 98 to evaluate whether reinforcement grades higher than 60 could be used for the primary reinforcement in members that are part of the seismic-force-resisting system [1]. The ATC-98 project team developed a list of potential issues related to the use of high-strength reinforcement, searched for and reviewed relevant literature, performed studies to evaluate what changes in the response of structures and members could be expected with the use of high-strength reinforcement, and studied potential cost savings that could result from the use of Grade 80 reinforcement. This paper provides a brief overview of the ATC-98 project findings. A similar summary may be found in an earlier paper that was written just prior to completion of the final ATC-98 report [16].

HIGH-STRENGTH REINFORCEMENT AVAILABILITY AND CHARACTERISTICS

In this paper, high-strength reinforcement is defined as reinforcement with minimum specified yield strength of at least 75 ksi. High-strength reinforcement types meeting this criterion that are available for purchase in the U.S. include ASTM A706 Grade 80, ASTM A1035 Grades 100 and 120, and SAS670 with specified yield strength of 97 ksi. High-strength stainless steel reinforcement is also available but it is not considered in this paper. ASTM A706 Grade 80 reinforcement is the only high-strength

[1] Principal with Simpson Gumpertz & Heger, Inc., Waltham, Massachusetts, USA
[2] Professor at the University of Kansas, Lawrence, Kansas, USA
[3] Principal with Mar Structural Design, Berkeley, California, USA
[4] Professor at the University of California, San Diego, La Jolla, California, USA
[5] Senior Vice President with Charles Pankow Builders, Ltd., Pasadena, California, USA
[6] Associate with KPFF Consulting Engineers, Seattle, Washington, USA

reinforcement available in the U.S. that is specifically intended for use as flexural reinforcement in members resisting earthquake effects.

Although not available in the U.S., Japan's SD685, also referred to as USD685, with a specified yield strength of approximately 100 ksi and New Zealand's 500E with a specified yield strength of approximately 75 ksi are considered in this paper because they were specifically developed for use in seismic design. Japan's SD685 reinforcement was developed as part of Japan's New RC Project that was undertaken to develop provisions for the design of tall structures using high-strength materials [6].

Different properties and definitions of properties are used in various specifications of reinforcement. Tensile properties and other requirements defining strength and ductility that might be specified are listed in the first column of Table 1. Table 1 also shows specified properties for six types of reinforcement discussed in this paper. Some, but not all, of these properties and tests are specified for each reinforcement type. For example, although ASTM A706 Grade 60, Grade 80, and AS/NZ 500E reinforcement typically exhibit a yield plateau, Table 1 shows that SD685 is the only type of reinforcement that has a specified elongation at the end of the yield plateau. Similarly, AS/NZ 500E is the only reinforcement in Table 1 that has a required minimum uniform elongation.

Table 1. Tensile properties specified for high-strength reinforcement.

	A70 Grade 60	**A706 Grade 80**	**A1035 Grade 100**	**SAS670**	**SD685**	**AS/NZ 500E**
Min. yield strength, f_y	60 ksi	80 ksi	100 ksi	97 ksi[1]	100 ksi	72 ksi[1]
Max. yield strength	78 ksi	98 ksi	N.A.	N.A.	114 ksi	87 ksi
Min. tensile strength, f_u	80 ksi	100 ksi	150 ksi	116 ksi	N.A.	N.A.
Min. tensile strength to actual yield strength ratio	1.25	1.25	N.A.	1.10	1.18	1.15[2]
Min. total elongation in 8 in.[3]	10 to 14%	10 to 12%	6% to 7%	6% to 9%	10%	N.A.
Uniform elongation, A_{gt}	N.A.	N.A.	N.A.	N.A.	N.A.	10%
Min. elongation at end of yield plateau	N.A.	N.A.	N.A.	N.A.	1.4%	N.A.

1. Based on 5% fractile value.
2. Maximum actual tensile strength to actual yield strength ratio is 1.4.
3. The minimum elongation is shown as a range because different minimums apply for different bar sizes.

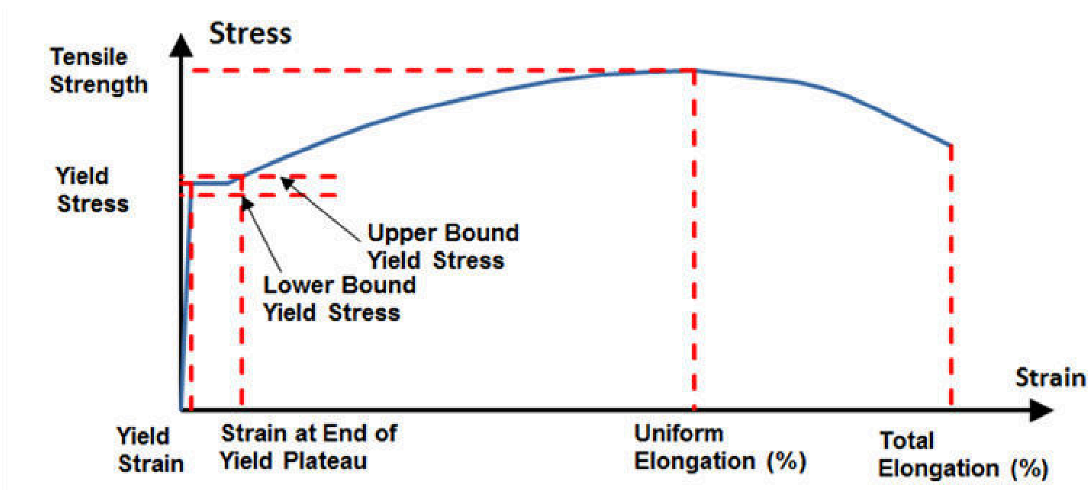


Figure 1. Idealized stress-strain curve with various tensile characteristics [1].

Several of the tensile properties listed in Table 1 are defined on an idealized stress-strain curve shown in Fig. 1. It can be seen in Fig. 1 that the uniform elongation occurs at the peak tensile strength. At the uniform elongation, a bar experiences uniform strain along its length (i.e., at the onset of necking). Because of cumulative damage during reversed cyclic straining, a bar in a critical region of a seismic resisting element may rupture at an elongation smaller than the uniform elongation. For this reason, some codes [3,4,5,17] limit the maximum strain that can be used in the design of critical yielding regions of structures to between 60 and 80 percent of the uniform elongation.

Based on the reviews and studies performed by the ATC-98 project team, recommendations for reinforcement tensile properties were developed. These are shown in Table 2. Quasi-static cyclic reversed load testing as well as shake table tests have shown that, where plastic hinges develop, the presence or lack of a yield plateau in the longitudinal reinforcement makes little difference in the response of the element or system. Therefore, a yield plateau is not required. Another recommendation is that uniform elongation should be reported rather than total elongation. This is supported by the fact that some U.S. codes already make use of the uniform elongation in design requirements [3,4,5].

Table 2. Recommended properties of Grade 100 and stronger reinforcement.

Property or Characteristic	Recommendation
Maximum yield strength	115% of specified minimum yield strength
Minimum measured tensile-to-measured yield strength ratio	1.2
Minimum uniform elongation	8%
Minimum total elongation[1]	12%

1. Total elongation is not required if uniform elongation is reported.

DEFORMATION CAPACITY OF BEAMS, COLUMNS, AND WALLS

Cyclic test results of beams, columns and walls with high-strength reinforcement (available in the U.S.) were studied to determine whether performance equivalent to that of members with Grade 60 reinforcement is achieved.

Beams

The cyclic response of cantilever concrete beams reinforced with high-strength steel bars was studied by Tavallali [7]. The tensile strength and total elongation of the steel bars were 98 ksi and 16% for the Grade 60 bars and 117 ksi and 10% for the Grade 97 bars. Fig. 2 shows the reinforcement details and specimen dimensions. Lateral force vs. drift results for two beam specimens, CC4-X with Grade 60 reinforcement and UC4-X with Grade 97 reinforcement are presented in Fig. 3.

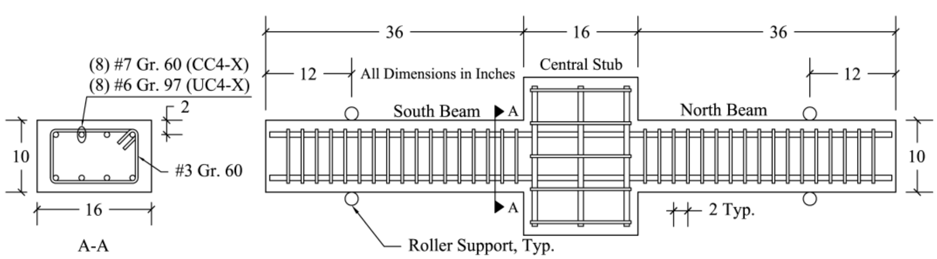
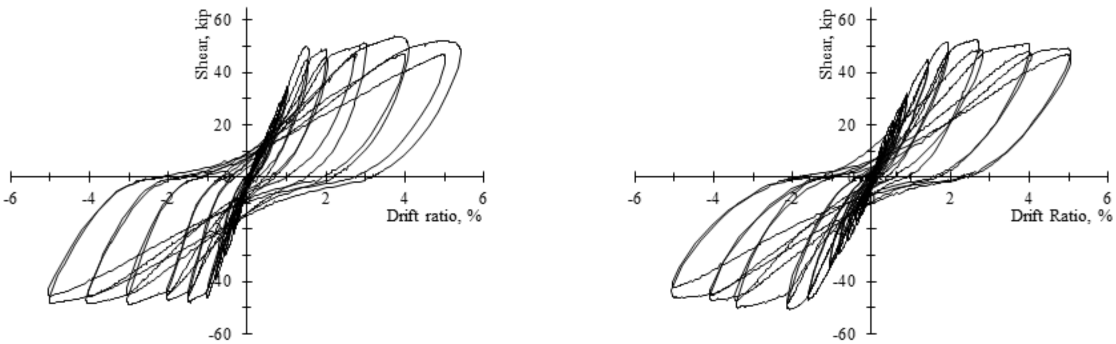


Figure 2. Reinforcement details for beam specimens [7].



Specimen CC4-X with Grade 60 bars Specimen UC4-X with Grade 97 bars

Figure 3. Measured shear versus drift ratio for beams [7].

The measured shear-drift response for CC4-X is shown on the left in Fig. 3. The specimen tolerated the two cycles of 5% drift while maintaining a load carrying capacity similar to the peak load resisted in previous cycles. The measured shear-drift response for UC4-X is shown on the right in Fig. 3. Compared to Specimen CC4-X, Specimen UC4-X showed reduced post-cracking stiffness and increased yield deformation. The specimen tolerated the two cycles of 5% drift while maintaining a load carrying capacity similar to the peak load resisted in previous cycles. Both beam specimens, CC4-X and UC4-X, had similar behavior. Specimen CC4-X showed a small increase in shear after yielding while specimen UC4-X had a nearly flat post-yield shear-drift curve, which resembled the stress-strain curve of the Grade 97 bars. The conventional Grade 60 bars were characterized by a tensile-strength-to-yield-strength ratio (f_u / f_y) of 1.5, while the Grade 97 bars had a ratio of 1.2. The beam test data indicate that replacing Grade 60 longitudinal reinforcement with reduced area of Grade 97 reinforcement leads to comparable flexural strength and deformation capacity.

Columns

Restrepo et al. [8] tested two circular cantilever bridge columns. One of the columns, Unit 1, was built with Grade 60 ASTM 706 longitudinal and transverse reinforcement and the second column, Unit 2, was built using Grade 100 ASTM 1035 longitudinal and transverse reinforcement. The 35% scale columns were 3 ft in diameter and 9.5 ft tall. Unit 1 had two cages, each containing 42 No. 5 bars tied to No. 3 fuse-welded hoops spaced at 1.56 in. on centers. The longitudinal reinforcement ratio, ρ_ℓ, for Unit 1 was 2.54% and the volumetric transverse reinforcement ratio was ρ_s = 1.74%. Unit 2 had a single cage with 42 No. 5 longitudinal bars tied to No. 3 fuse-welded hoops spaced at 1.56 in. on centers. The ASTM 1035 longitudinal bars for Unit 2 had a yield strength of 94 ksi, a tensile strength of 155 ksi, and a uniform elongation of 5.1%. Both units were tested with an axial load of about $0.07f_c'A_g$. Fig. 4 shows the response of the two specimens.

Unit 1 was cycled to drift ratios in excess of 6% when extensive yielding of the hoops led to longitudinal bar buckling followed by fracture. Unit 2 was tested through three complete cycles at a drift ratio of 3.9% with only spalling of the concrete cover. The hoops had very effectively restrained the longitudinal bars from buckling up to this drift level. Degradation in the response began with the fracture of a hoop at 5 in. above the base of the column. This hoop fractured at 3.1% drift on the first cycle to a target drift of 6%. The hoop fractured in the heat-affected zone adjacent to the fuse weld. Unit 1 showed fatter hysteresis loops and a higher effective stiffness to the yield point than Unit 2. However, Unit 2 showed significantly smaller residual displacements than Unit 1 after being displaced to the same drift ratio. The results give a positive indication that high-strength reinforcement can be used successfully in columns up to drift levels of about 4%, for the range of axial loads evaluated.

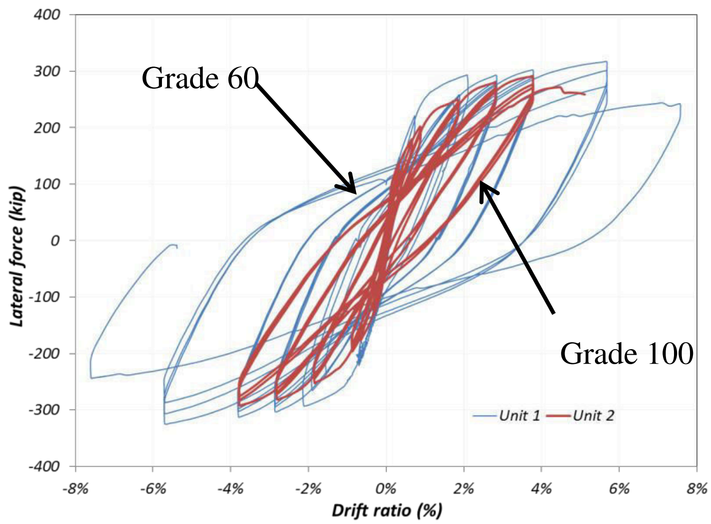


Figure 4. Hysteretic response of Units 1 and 2 tested (Courtesy of Restrepo).

Rautenberg [9] tested concrete columns reinforced with Grade 120 longitudinal bars and compared the results to tests with Grade 60 bars. Two tests are described here. The tensile strength and total elongation were 92 ksi and 20% for the Grade 60 bars in specimen CC-3.3-20 and 168 ksi and 8.6% for the Grade 120 bars in specimen UC-1.6-20. Specimen UC-1.6-20 had approximately one-half the area of longitudinal reinforcement as specimen CC-3.3-20 so the area of steel times the yield strength $A_s f_y$ was nearly identical. Typical geometry and reinforcement details of the test specimens are shown in Fig. 5.

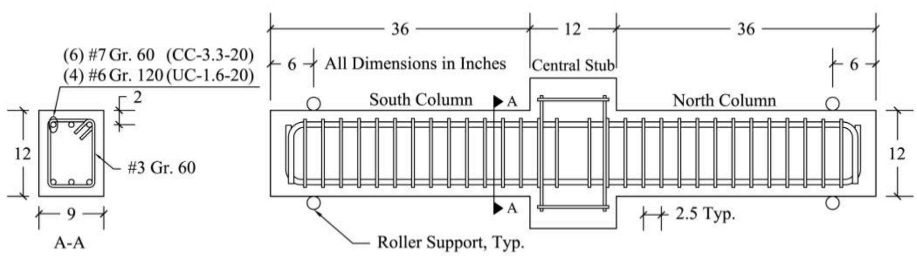
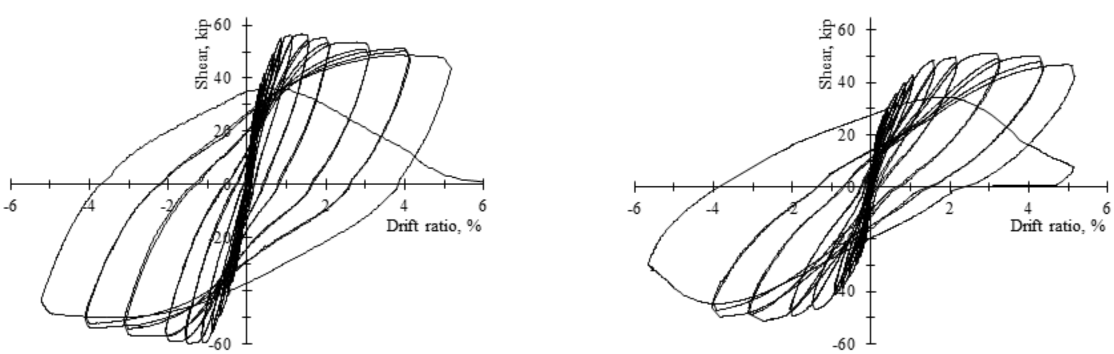


Figure 5. Reinforcement details for column specimens [9].



(a) Specimen CC-3.3-20 with Grade 60 bar

(b) Specimen UC-1.6-20 with Grade 120 bar

Figure 6. Measured shear versus drift ratio for columns [9].

Experimental shear versus drift ratio data are shown in Fig. 6 for column specimens CC-3.3-20 and UC-1.6-20. Both specimens were subjected to displacement reversals while carrying a constant axial load of 0.2 $f_c'A_g$. CC-3.3-20 completed the first cycle to 5% drift as shown on the left in Fig. 6, but the longitudinal bars buckled during the second cycle to that drift ratio. Specimen UC-1.6-20 with Grade 120 bars completed the first half-cycle to 5% drift but the longitudinal bars buckled during the second half-cycle to that drift ratio. Testing was continued and the remaining longitudinal bars buckled at a small drift ratio (approximately 2%) during the second cycle to 5% drift. Compared with Specimen CC-3.3-20, Specimen UC-1.6-20 showed reduced post-cracking stiffness and increased yield deformation. The column test data indicate that replacing conventional Grade 60 longitudinal reinforcement with reduced amounts of Grade 120 reinforcement leads to comparable flexural strength and deformation capacity, within the range of axial loads tested.

Structural walls

In the U.S., structural walls, also referred to as shear walls, are typically rectangular in shape, part of a core wall system with coupling beams and piers, or are configured to create cross sections made up of several walls (e.g., C-, L-, T-shaped) to accommodate banks of elevators or stairs. The tests of walls with high-strength reinforcement that are most relevant to U.S. practice are three 1/5-scale wall specimens studied by Kimura and Ishikawa [10]. The longitudinal bars for the boundary elements and the typical horizontal and vertical wall bars were grade SD685 with a nominal yield strength of 100 ksi. The shear span-to-depth ratio of the specimens was 2.0, and the walls were rectangular, 5.9-in. thick and 59-in. long. Other characteristics of the wall specimens are listed in Table 3. The area of transverse reinforcement did not comply with the requirements in ACI 318 for special boundary elements, but other details of the reinforcement did comply. The transverse reinforcement spacing to longitudinal bar diameter ratio, s/d_b, was 5.

The ultimate drift was 1.5% for the wall with an axial load of 0.15 $f'_c A_g$ and 2% for the walls with an axial load of 0.10 $f'_c A_g$. All walls exhibited a flexural compression failure mode in which the concrete crushed and the bars buckled. Given the limited number of applicable tests of flexural-controlled wall specimens with high-strength reinforcement, additional testing is recommended to verify that walls with high-strength reinforcement provide performance that is equivalent to walls with Grade 60 reinforcement.

Table 3. Characteristics of wall specimens [10].

Specimen	**07N10**	**07N15**	**10N10**
Measured concrete strength, f'_c (ksi)	10.9	10.9	15.8
Axial stress ratio, $P/(f'_c A_g)$	0.10	0.15	0.10
Longitudinal bars[1] at boundary element	14-D13 (SD685 with measured f_y = 102 ksi), ρ_{be} = 3.94%		
Transverse reinforcement[1] at boundary elements	6-D6 @ 2.6 in. (SD685 measured f_y = 109 ksi) ρ_{vol} = 1.39%		
Vertical and horizontal reinforcement[1]	Two layers of D10 @ 3.9 in. (SD685, measured f_y = 114 ksi), $\rho_t = \rho_\ell$ = 1.14%		

1. Area of a 13 mm deformed bar (D13) is 0.196 in.^{2}; Area of a 10 mm deformed bar (D10) is 0.11 in.^{2}; Area of a 6 mm deformed bar (D6) is 0.049 in.^{2}.

DEPTH OF INTERIOR BEAM-COLUMN JOINTS OF MOMENT FRAMES

The ACI 318 code requires that the depth of beam-column joints (the horizontal dimension of the joint) must be 20 times or larger than the diameter, d_b, of the longitudinal horizontal bars passing through the joint. This requirement is based on research by Zhu and Jirsa [11]. Lin et al. [12] tested joints with longitudinal bars that had yield strengths of 75 ksi, and Aoyama [6] presents the results of tests with longitudinal bars that had yield strengths of 60 to 100 ksi. Based on the research reviewed, Eq. 1 is proposed as a simplified equation for calculating beam-column joint depths.

$$h = \frac{1}{1{,}200} \frac{f_y^{1.3}}{\sqrt{f_c'}} d_b \tag{1}$$

where h is the depth of the joint (inches), f_y is the specified yield strength (psi) of the longitudinal bar, f'_c is the concrete strength (psi), and d_b is the diameter of the longitudinal horizontal bar passing through the joint in inches. Alternatively, and following the format of the equation for the joint depth in ACI 318, $26d_b$ is proposed for Grade 80 reinforcement and $35d_b$ is proposed for Grade 100 reinforcement. Joint depths (i.e. column widths) of $26d_b$ and $35d_b$ are typically not feasible in most commercial structures because the columns are excessively large. Therefore, the authors recommend that additional tests be performed to confirm the validity of these recommendations, and to explore alternative mechanical devices that may be used to prevent bar slip through the joints without increasing column widths.

BUILDING DRIFTS

Members with high-strength bars can achieve the same design strength as conventional members while using less longitudinal reinforcement, but the reduction in the area of longitudinal reinforcement causes a reduction in the post-cracking stiffness. The reduction in the post-cracking stiffness has an impact on the effective flexural rigidity of the section that is typically used in conventional linear-elastic structural modeling.

To evaluate the impact of the reduction in longitudinal reinforcement caused by the use of high-strength steel, a number of parametric, nonlinear, 3-D, response-history analyses were performed on a 13-story case study building. This building was based on the reinforced concrete frame-wall design example from FEMA 451 [13]. The details of the structure are found in Barbosa [14]. The analyses assessed the sensitivity of building roof and interstory drifts to reinforcement yield strength. The building was redesigned using reinforcement grades of 60 to 100 ksi. The maximum increase in drift for Grade 100 reinforcement compared to Grade 60 reinforcement was approximately 20%.

BAR BUCKLING RESTRAINT

ACI 318 limits the spacing of transverse reinforcement in potential plastic hinges of beams and columns and on boundary elements of walls to six times the diameter of the longitudinal bar ($6d_b$). This requirement is aimed at restraining the longitudinal reinforcement, to delay buckling when yielding and hardening occur in tension and compression in the plastic hinge region of an element. In the ATC-98 Report [1], a discussion of references regarding bar buckling is presented, and an analytical study that was performed as part of the ATC-98 research is summarized. Based on the results of the reviews and analytical study, as well as consideration of bar over-strength and construction tolerances, maximum transverse bar spacing of $5d_b$ for Grade 80 reinforcement and $4d_b$ for Grade 100 reinforcement are recommended.

LAP-SPLICE AND DEVELOPMENT LENGTHS

Seliem et al. [15] compared the stress levels developed in tests of lap-spliced bars to stress levels computed using development length equations calculated from ACI 318. The results of lap-splice tests without transverse reinforcement, referred to as unconfined lap-splice tests, indicate that the ACI 318 design limits for development length and Class A lap-splices require modification for high-strength reinforcement. Although not presented in the paper [15], applying a 1.3 factor to the development length equation (similar to the requirements for a Class B lap splice) reduces but does not eliminate the need for modification of splice lengths for high-strength reinforcement. Further modification is required in the form of an adjustment to the requirement for confining reinforcement adjacent to the splice. Fig. 7 shows the ratio of developed-to-calculated stress on the vertical axis versus the ACI 318 confinement term, K_{tr}, on the horizontal axis. A minimum recommended value of K_{tr} for splices and critical development lengths within members of special moment frames and structural walls is 1.3.

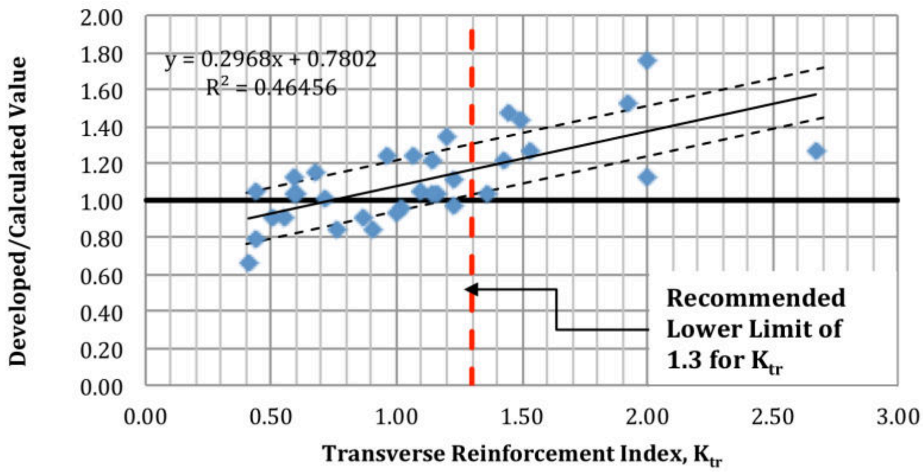


Figure 7. Developed-to-calculated stress values versus transverse reinforcement index, K_{tr}.

CONCLUSIONS

Based on the research reviewed and the studies performed, the authors conclude that it is feasible to design reinforced concrete members to resist earthquake effects using reinforcement with yield strengths of 80 ksi and higher. At this time, there is insufficient information available to recommend code changes to allow the broad use of reinforcement stronger than Grade 80, although the information currently available points favorably to this possibility in the future.

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