ATC 115

Roadmap for the use of high-strength reinforcement in reinforced concrete design

ATC Applied Technology Council

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Applied Technology Council

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Cover illustration: Placement of reinforcement in and around the coupling beam of a special reinforced concrete shear wall (courtesy of Ron Klemencic).

ATC-115

Roadmap for the Use of High-Strength Reinforcement in Reinforced Concrete Design

by

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Preface

In 2012, the Charles Pankow Foundation (CPF) began to investigate the feasibility of incorporating reinforcing steel in excess of 60 ksi into the ACI 318 Building Code. This investigation was prompted by interest on the part of structural engineering practitioners, structural concrete constructors, and key academic researchers who felt that the use of higher strength reinforcing bars could provide a significant benefit to the industry. Initial investigative efforts by CPF first included informal meetings with an expert panel, followed by the commissioning of several research projects studying the technical feasibility of using higher strength bars as well as developing a technical definition of the product. At the same time, CPF engaged steel reinforcing bar producers to evaluate the technical and financial feasibility of making high-strength bars commercially available.

By mid-2013, initial studies confirmed the technical feasibility of using high-strength reinforcement in design. In addition, reinforcing bar producers verified that higher strength bars could be manufactured and made available through normal distribution channels. Also in 2013, in an unrelated effort funded by the National Institute of Standards and Technology (NIST), the ATC-98 Project completed the GCR 14-917-30 report, *Use of High-Strength Reinforcement in Earthquake-Resistant Concrete Structures* (NIST, 2014). This report confirmed the feasibility of using high-strength reinforcement in seismic applications.

The last comprehensive update to ACI 318 related to reinforcing bar strength was made in 1971. With technical feasibility and manufacturing capability confirmed, the next task was to determine the research necessary to support an update to ACI 318 to incorporate the general use of reinforcement in excess of 60 ksi. CPF commissioned the development of a roadmap that effectively reviewed every applicable section of ACI 318-14, and identified new research or engineering studies necessary to support such a change. With a long history of successful collaboration on research projects, CPF selected the Applied Technology Council (ATC) to manage this process. Entitled *The Development of a Roadmap on Use of High-Strength Reinforcement in Reinforced Concrete Design*, this project can be found on the CPF website (www.pankowfoundation.org) as Research Grant Agreement #05-13, also known as the ATC-115 Project.

CPF wishes to acknowledge the hard work of Jon A. Heintz, Project Manager for ATC. CPF is indebted to the leadership of Dominic J. Kelly, Project Technical Director, and to the members of the Project Management Committee, consisting of David Darwin, David C. Fields, Robert J. Frosch, Andres Lepage, Joseph C. Sanders, and Andrew S. Whittaker, for their contributions in developing this report and the resulting recommendations. The Project Review Panel, consisting of Wassim Ghannoum, S.K. Ghosh, Ramon Gilsanz, James O. Jirsa, Mike Mota, Thomas C. Schaeffer, and Loring A. Wyllie, Jr., provided technical review and commentary at key developmental milestones during the project. CPF also wishes to acknowledge the contributions of Conrad Paulson and Jack Moehle as special consultants to CPF, and Robert Risser (Chief Executive Officer of CRSI) along with the many CSRI producers and members who contributed to this effort. Finally, CPF wishes to thank its Board of Directors, Richard Kunnath, Ron Klemencic, and Tim Murphy for their support and contributions to this effort.

The completion of the ATC-115 Project is an important milestone in the process of incorporating high-strength reinforcement into building design and construction practice. The research effort outlined in this report will be one of the largest research programs ever undertaken in the reinforced concrete industry, and implementing this program will require the support of the entire industry. The net result, however, will be much broader than the addition of high-strength reinforcement into ACI 318. Research, testing, and engineering studies investigating and justifying the use of high-strength reinforcement will have the added benefit of bringing most of the technical specifications in the ACI 318 Building Code into the 21st Century.

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Chapter 1
Introduction

Concrete reinforcement with a yield strength greater than 75 ksi is becoming more available in the United States. In 2009, the American Society for Testing and Materials (ASTM) International included Grade 80 in ASTM A615, *Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement* (ASTM, 2009a), which is the most commonly referenced specification for reinforcing bars. Since 2004, ASTM A1035, *Standard Specification for Deformed and Plain, Low-Carbon, Chromium, Steel Bars for Concrete Reinforcement* (ASTM, 2014), has included Grade 100 reinforcing bars, and Grade 120 bars were added in 2007. A German-manufactured bar with a minimum yield strength of 97 ksi, SAS 670 (EOTA, 2013), has an International Code Council (ICC) Evaluation Services (ES) report for use in the United States (ICC-ES, 2011). Along with ASTM A1035 Grade 100 reinforcing bars, SAS 670 has been approved for use as column reinforcement by the New York City Department of Buildings. In the near future, many producers in the United States will be capable of producing Grade 100 reinforcement with mechanical properties suitable for general design applications, and similar availability of Grade 120 reinforcement is likely to occur in the United States within 10 years.

1.1 Purpose and Scope

The primary objective of the ATC-115 Project was to prepare a detailed *Roadmap* that specifically identifies the technical support required, whether it be the results of new research, engineering studies, or re-evaluation of existing research findings, to effect updates of ACI 318-14, *Building Code Requirements for Structural Concrete and Commentary* (ACI, 2014a), to allow the general use of steel reinforcement in excess of Grade 80 for gravity and wind load applications, and in excess of Grade 60 for special seismic systems. In the context of this work, high-strength reinforcement refers to reinforcement with grades higher than Grade 60. Specifically, the use of Grade 80, Grade 100, and Grade 120 reinforcement was considered for applications in members designed for gravity loads, wind loads, or seismic loads, as part of a gravity system, or as part of an ordinary, intermediate, or special seismic-force-resisting system.

The resulting *Roadmap* outlines the steps needed to:

- further investigate the use of high-strength reinforcement in general reinforced concrete design and construction in building and non-building structural applications; and
- develop code-change proposals and encourage adoption of high-strength reinforcement in the code development process.

Development of this *Roadmap* was based on the following activities:

- identification of production, fabrication, and design issues to be considered;
- identification of constructability challenges;
- search for relevant research and other available information on high-strength reinforcement in the published literature;
- determination of the current state-of-knowledge regarding issues and challenges associated with high-strength reinforcement;
- evaluation of the current state-of knowledge and determination of when existing information was sufficient or when additional research or study was needed;
- identification of additional experimental research and engineering studies needed; and
- estimation of the approximate budget, schedule, and prioritization for a recommended research and study program.

1.2 History of High-Strength Reinforcement and ACI 318

American Concrete Institute (ACI) Committee 318 is responsible for updating and maintaining its *Building Code Requirements for Structural Concrete and Commentary* (ACI 318). Over the years, ACI 318 has been updated to include provisions for higher strength reinforcement as it became available, and when there was sufficient behavioral data to justify its use. In the 1950s and early-1960s, Intermediate Grade (Grade 40) and Hard Grade (Grade 50) reinforcement had been available for about 50 years, and was commonly used. In 1959, ASTM specifications A432 (ASTM, 1959a) and A431 (ASTM, 1959b) were published, which introduced Grade 60 and Grade 75 reinforcement, respectively. The 1963 version of ACI 318 allowed the use of steel bars with a yield strength of 60 ksi. In 1968, ASTM A615, which included Grades 40, 60, and 75, was introduced.

In the late-1950s and 1960s, the Portland Cement Association (PCA) conducted a series of tests reported in eight parts that examined beams, girders, and columns (Hognestad, 1961; Hognestad, 1962; Gaston and Hognestad, 1962; Kaar and Mattock, 1963; Pfister and Mattock, 1963; Pfister and Hognestad, 1964; Kaar and Hognestad, 1965; and Kaar, 1966). These tests covered flexural strength, control of flexural cracking, compression splices in columns, and fatigue. Reinforcement strengths ranged from 55 ksi to 120 ksi. At about the same time, Thomas and Sozen (1965) published the results of tests of beams reinforced with unstressed prestressing reinforcement with a yield strength of 230 ksi. These early tests were considered in the 1971 edition of ACI 318, when the upper limit for yield strength was increased to 80 ksi. At the time, there were no ASTM standard specifications for reinforcement with yield strengths greater than 75 ksi.

In ACI 318-71, the maximum specified yield strength was restricted to 60 ksi for reinforcement in special seismic systems, and this limit is still in effect in ACI 318-14. The Structural Engineers Association of California (SEAOC) developed a specification for reinforcement with more restrictive tensile properties and chemistry controls, published as ASTM A706, *Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement* (ASTM, 1974). ASTM A706 was permitted in ACI 318-77. In ACI 318-83, ASTM A706 was required for special seismic systems, but ASTM A615 was permitted if specified mechanical properties were met.

The 2009 versions of ASTM A615 and A706 specifications (ASTM, 2009a; ASTM, 2009b) were the first to include requirements for Grade 80 reinforcement. ACI Committee 318 adopted these specifications without restriction in the main body of ACI 318-11 because the use of Grade 80 reinforcement was already permitted. However, Grade 80 reinforcement is currently not permitted for use in special moment-resisting frames and special structural walls due to insufficient test data for cyclically loaded members with Grade 80 reinforcing bars.

1.3 Prospects for Adoption of High-Strength Reinforcement into ACI 318

Significant research was completed in Japan as part of the New RC Project (Aoyama, 2001), which took place between 1988 and 1993. Research has continued in Japan, and is being performed in Taiwan, Korea, and the United States. Much of this research has been identified and described in NIST GCR 14-917-30, *Use of High-Strength Reinforcement in Earthquake-Resistant Concrete Structures* (NIST, 2014).

High-strength reinforcement has been used in hundreds of high-rise buildings in Japan, and is gaining traction in U.S. design and construction practice. Documents such as ACI ITG-6, *Design Guide for the Use of ASTM A1035/A1035M Grade 100 Steel Bars for Structural Concrete* (ACI, 2010a), and NCHRP Report 679, *Design of Concrete Structures Using High-Strength Steel Reinforcement* (Shahrooz et al., 2011), have made progress towards identifying how some code provisions in ACI 318 and the American Association of State Highway and Transportation Officials (AASHTO) *LRFD Bridge Design Specifications* (AASHTO, 2014) could be changed to incorporate high-strength reinforcement.

Adoption of high-strength reinforcement into ACI 318 will require a substantial effort because there are many issues associated with the use of high-strength reinforcement that will need to be addressed, and many sections of the code will require new or revised provisions. Research and engineering studies identified in this *Roadmap* are intended to develop the necessary code change proposals, and to provide the information and comprehensive data that are needed in support of these changes.

1.4 Prospects for Adoption of High-Strength Reinforcement into other U.S. Codes and Standards

The nuclear industry is interested in using high-strength reinforcement in new construction to reduce congestion, improve the quality of placed concrete, and speed construction time. Adoption of high-strength reinforcement into ACI 349-13, *Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary* (ACI, 2014b), has been discussed by ACI Committee 349. Potential reduction in minimum elongation associated with the use of higher strength reinforcement is of lesser concern in the nuclear industry because component nonlinear deformation demands are much smaller than those in buildings and bridges at design level shaking intensities.

The bridge industry is also interested in using high-strength reinforcement in new construction. Goals for the use of high-strength reinforcement are similar to those of the nuclear industry. The California Department of Transportation is currently working with North Carolina State University to provide strain information on ASTM A706 Grade 80 reinforcement. If adequate ductility is shown, ASTM A706 Grade 80 reinforcing bars could be used in capacity-protected components, such as bent caps, where rebar congestion is the highest. However, for use in seismic members that are expected to yield (e.g., columns), rigorous testing of couplers and splices will be necessary. Other State Departments of Transportation (DOTs), and the

Federal Highway Administration (FHWA), have also shown significant interest in high-strength reinforcement for general reinforced concrete design.

1.5 Key Issues Associated with the Use of High-Strength Reinforcement

Key issues associated with the use of high-strength reinforcement in the design and construction of reinforced concrete buildings and non-building structures stem from bar production and fabrication challenges, design requirements, and impacts on constructability. These issues are described in Chapter 2 and Chapter 3 of this *Roadmap*.

1.6 Report Organization and Content

This *Roadmap* identifies key issues in production, fabrication, design, and construction related to the use of high-strength reinforcement, identifies the current state-of-knowledge, and outlines a recommended program of experimental research and engineering studies necessary to investigate the use of high-strength reinforcement and support code change proposals for adoption of high-strength reinforcement in building codes and standards for reinforced concrete design.

Chapter 2 identifies the current state-of-knowledge on issues related to production and fabrication of high-strength reinforcement, and construction using high-strength reinforcement.

Chapter 3 identifies the current state-of-knowledge on issues related to design using high-strength reinforcement.

Chapter 4 outlines recommended experimental research and engineering studies generated by the issues identified in Chapter 2 and Chapter 3, which is needed to investigate the use of high-strength reinforcement and support the development of code change proposals.

Chapter 5 provides a summary of the overall program, along with estimated budget requirements and recommendations for prioritization, scheduling, and implementation of the program.

Chapter 2
Production, Fabrication, and Construction Issues

The production and fabrication of high-strength reinforcement will be influenced by the mechanical properties required for use in general design applications, and the resulting impacts on the cost of production. This chapter describes production, fabrication, and construction issues associated with the use of high-strength reinforcement.

2.1 Reinforcement Production and Specification Issues

Reinforcement used in general design applications must have properties that can be shown to result in structures that are safe and serviceable. Basic mechanical properties that are important for achieving safe and serviceable designs include: yield strength, tensile-to-yield strength ratio, elongation at tensile strength, and length of yield plateau. In currently available high-strength reinforcement, higher yield strength is often accompanied by reductions in the tensile-to-yield strength ratio, uniform elongation (elongation at tensile strength), and length of yield plateau. In some cases, the yield plateau is completely eliminated.

The required mechanical properties of high-strength reinforcement, and the means by which reinforcing bars will be specified, will be established as part of the program described in this *Roadmap*. Issues related to fabrication, inventory and storage, identification of bars on the site, production costs, and required properties for various design conditions will affect the criteria used for specifying high-strength reinforcement. It is recognized that the mechanical properties eventually selected for achieving safe and serviceable designs must be compatible with the capabilities of producers and the economics of reinforcing bar production.

2.1.1 Production of Deformed High-Strength Reinforcement

The means by which high-strength reinforcement will be produced are expected to depend on the mechanical properties that are needed. Reinforcement used under gravity load conditions (i.e., dead, live, ice, snow, and rain) requires reliable yield strength with less emphasis on ductility, whereas reinforcement used under seismic load conditions requires more emphasis on elongation and tensile-to-yield strength ratio. Maximizing the benefits of using high-strength reinforcement will likely involve minimizing changes to current methods of producing reinforcing bars. As technology advances, it is anticipated that the methodologies for producing high-strength reinforcement will also evolve.

Current methods for increasing yield strength include quenching and tempering, micro-alloying, and work-hardening. Quenching and tempering involves the use of water to rapidly cool steel that has been heated to the austenite phase (the phase at which solid steel recrystallizes), followed by self-tempering resulting from the gradual release of the heat that is trapped in the core of the quenched steel. Quenching results in a hard metal structure while self-tempering softens the steel and increases its toughness.

Reinforcement must be rolled in a malleable state so that bar deformations will form properly. Therefore, reinforcing steel is rolled while it is still red hot. In the quenching and self-tempering process, bars are sprayed with water just after rolling, producing an outer layer of martensite (hard phase). The bar speed and the amount of water are carefully controlled to leave the core of the bar unquenched. The hot core then tempers the quenched outer surface, resulting in a bar with high strength and some degree of ductility. This process improves the bendability and fatigue resistance of the bar. When subjected to tension, residual stresses in the bar cause yielding of the core prior to the outer surface, resulting in a stress-strain curve with a rounded shape, and without a sharp yield point.

Less grain refinement is achieved by rolling larger bars, so micro-alloying is often used to achieve desired properties. Micro-alloying involves the addition of small quantities of certain alloying elements in the molten steel to induce grain refinement. Micro-alloying is commonly used for most Grade 80 reinforcement produced in the United States, as well as for large Grade 60 reinforcement (i.e., No. 9 and larger). Generally, micro-alloyed bars have a higher tensile-to-yield strength ratio than quenched and tempered bars, and a lower tensile-to-yield strength ratio than plain carbon steel bars.

In New Zealand, both quenching and tempering and micro-alloying are used to produce ductile high-strength reinforcement (on the order of 75 ksi) specifically for use in earthquake-resistant structures. The combination of these two processes has the potential for producing even higher strength reinforcement.

Work-hardening involves the plastic deformation of reinforcing steel at or near room temperature. Cold drawing plain and deformed wire reinforcement is a form of work-hardening. In some countries, the strength of bars is increased by the process of cold twisting after hot rolling. Work-hardening increases the yield strength but eliminates the yield plateau, and reduces both the achievable elongation and the tensile-to-yield strength ratio. Work-hardening may be suitable for production of reinforcement used in members where yielding is not expected. For current design practice in the United States, work-hardening is likely not a suitable method of production for reinforcement used in earthquake-resistant structures, where yielding is expected.

Reinforcing bars are most commonly produced in straight lengths at the mill. However, coiling smaller sized bars (i.e., No. 3 to No. 6) is becoming more common, and currently represents nearly 5% of the market for bars of all sizes. Bars are coiled soon after rolling, which traps heat in the coil. Thus, the cooling rate of coiled bars is somewhat slower than for straight bars. Because the samples must be straightened before testing, coiled bars tend to have a lower yield strength, and the shape of the stress-strain curve can be somewhat rounded. To counteract this effect, high-strength coiled bars will likely require higher quantities of micro-alloying elements as compared to corresponding sizes of straight bars.

In current manufacturing processes, bar identification marks are added during rolling, and mechanical properties are tested after the bars are rolled (and marked). If the mechanical properties of high-strength reinforcement are difficult to achieve, there is a risk that marked bars will not meet the specifications associated with the mark, and will need to be scrapped. Several alternatives for addressing this issue, ranging from the elimination of bar marks to the use of newer technologies for identifying and tracking bars, are possible:

- Elimination of bar marks – elimination of bar marks might avoid the need to scrap bars not meeting requirements for high-strength reinforcement, and might help avoid high-cycle and low-cycle fatigue failures that can occur at bar marks, but would not solve the need for identification and verification of bar grades at a job site.
- Laser printing – laser identification of bar heats would allow producers to distribute bars based on any compliant specification, but would require additional information in the field to allow verification of bar grade and placement at a job site.
- Color-coded ends of bars – color coding on the ends of bars to indicate grade is a possibility, but would not be effective in the case of bars that are shop cut before delivery to a job site.

- Laser printing after testing – would identify the actual grade after verification testing, but would require additional bar handling during the manufacturing process.

Requirements for bar elongation will also have an effect on methods for producing high-strength reinforcement. From an engineering perspective, reporting uniform elongation (i.e., the elongation that exists in the bar as the tensile strength is reached) is preferable to elongation at fracture, which is the increase in length over the 8-inch gage length including the necked-down region of the bar. At present, producers are considering the impacts of reporting uniform elongation. They do not have a history of reporting uniform elongation, so they are currently working to determine how uniform elongation is affected by chemistry and production processes.

Recommendations. Producers should take the lead in determining how high-strength reinforcement will be manufactured and produced. Input on the required mechanical properties is needed from the engineering and research communities so that safe and serviceable designs can be achieved. An initial test program, as described in Section 4.3.1, is recommended to explore issues related to the mechanical properties of high-strength reinforcement, and to assist in defining structurally acceptable properties for future development of ASTM standard specifications. Mechanical properties may vary with bar size. Although one or more producers have indicated a willingness to manufacture all bar sizes, production of a limited number of bar sizes in higher grades may be advantageous, at least initially.

2.1.2 Specification of High-Strength Reinforcement

As currently envisioned, the development of specifications for high-strength reinforcement is expected to involve the following:

- Expansion of ASTM A615 (carbon steel bars) to cover higher grades of reinforcement, possibly from 80 ksi to 120 ksi, in 20-ksi increments. This specification would not apply for reinforcement used in members of special seismic-force-resisting systems;
- Creation of a new ASTM specification covering all grades of reinforcement used in members of special seismic-force-resisting systems;
- Use of ASTM A706 (low-alloy steel bars) to cover only Grade 60 and Grade 80 reinforcement.

Producers have expressed an interest in having Grade 100 included in the ASTM A615 specification (or equivalent) in the near future. If ASTM A615

Grade 100 were to exist, many believe there would be demand for the higher grade reinforcing bars. Production of ASTM A615 Grade 100 reinforcement would provide an opportunity for experimentation and an experience basis for production of more ductile Grade 100 reinforcement with realistic material properties that could be included in a future specification targeted for use in earthquake-resistant construction (e.g., special moment frames and special structural walls).

Specification overlap should be maximized. For example, 100% of bars meeting the specification for seismic applications should be adequate for meeting the ASTM A615 specification, although not all ASTM A615 bars would be adequate for seismic applications. Overlap in specification requirements should occur at all strength levels, which will allow producers and fabricators to minimize bar inventories and storage requirements.

Specifications for bars with defined ranges of chemical composition, or for specific uses, are likely to be maintained, but with some modification. For example, Grade 100 and Grade 120 reinforcing bars are already produced under ASTM A1035. Changes might include adjustments to meet requirements for use in earthquake-resistant structures. Also, the use of high relative rib area deformations to improve bond characteristics could be considered, however, the potential reduced fatigue resistance of bars with high relative rib area would need to be explored.

Yield strength should be reported as the 0.2% offset (Paulson et al., 2013), as required in ACI 318-14, ASTM A615, ASTM A706, and ASTM A1035. For elongation, consideration should be given to reporting uniform elongation at peak stress rather than total elongation, as recommended in NIST GCR 14-917-30, *Use of High-Strength Reinforcement in Earthquake-Resistant Concrete Structures* (NIST, 2014).

The chemical restrictions in ASTM A706 (i.e., limits on carbon content to improve weldability) make it difficult to obtain high-strength steel bars with tensile-to-yield strength ratios and uniform elongations on the order of those routinely obtained for Grade 60 and Grade 80 reinforcement. As a result, including reinforcement higher than Grade 80 in ASTM A706 is not recommended at this time.

In some applications, reinforcement will be used in structures exposed to cold weather. With more stringent ductility requirements, ASTM A706 reinforcing bars are sometimes specified in such applications. Because ASTM A706 is not likely to include Grade 100 (or higher) reinforcement, supplemental requirements for high-strength reinforcement in cold-weather applications (e.g., Charpy V-notch toughness requirements) would need to be considered in the other specifications.

Recommendations. Producers should take the lead in determining how high-strength reinforcement will be specified. Input on the required mechanical properties is needed from the engineering and research communities so that safe and serviceable designs can be achieved using current design procedures. If the desired mechanical properties prove to be unattainable, design procedures could be adjusted based on mechanical properties that can be reliably achieved.

A detailed test program as described in Section 4.3.2 is recommended to determine mechanical properties for use in specifying high-strength reinforcement. In addition to tensile and bend tests, less common tests, such as low-cycle fatigue and strain-aging (bend-rebend) tests, should be performed. To evaluate the effects of bar bending on the ductile-to-brittle transition temperature of steel, consideration should be given to performing tests similar to those conducted by Hopkins and Poole (2005) on AS/NZS 4671, *Steel Reinforcing Materials* (AS/NZS, 2001), Grade 500E reinforcement. Other tests, such as Charpy V-notch toughness tests for reinforcement used in cold-weather applications should be included.

2.2 Fabrication Issues

Issues with fabrication of high-strength reinforcement can be grouped into the following two categories:

- the introduction of multiple grades of reinforcing bars that need to be scheduled, received, and stored at a fabrication facility prior to use; and
- changes in the fabrication process required as a result of the properties of high-strength reinforcing bars.

Expansion of product lines to include higher grade reinforcement will require more space to store multiple bar sizes of higher grade reinforcement prior to, and during, fabrication. The impact of this issue could be minimized through standardization around the use of fewer bar sizes in higher grades of reinforcement.

Fabrication processes of shearing and bending will be impacted by the properties of high-strength reinforcement. Higher grade reinforcement will result in higher shearing and bending forces for the same size bar, and will experience more elastic rebound after bending, leading to fabrication concerns regarding:

- wear and tear on existing equipment and the possible need for new, higher capacity equipment;
- safety of workers in the event of bar or equipment failure during bending operations; and
- compliance with bar fabrication tolerances.

Some shops have reported more frequent equipment failure associated with the fabrication of high-strength reinforcement. Safety concerns are heightened in cases where bar defects have caused fracture during bending operations at higher force levels. Extra precautions (e.g., worker protection cages) may be necessary to maintain a safe work environment, which could impact the efficiency of fabrication operations. The impact of this issue could be minimized through the use of larger bend diameters. However, changing the bend diameter will require additional research to investigate potential effects on the development length of standard hooks.

Tolerances for the fabrication of hooked bars are controlled in ACI 117-10, *Specification for Tolerances for Concrete Construction and Materials and Commentary* (ACI, 2010b). Increased elastic rebound of high-strength reinforcing bars will need to be considered in the fabrication process in order to meet current specified tolerances. Alternatively, adjusted tolerances for high-strength reinforcement could be considered.

Recommendations. Fabricators should take the lead on addressing concerns for wear and tear on equipment, safety of workers, and compliance with fabrication tolerances for high-strength reinforcing bars. In general, fabricating challenges associated with the use of higher strength reinforcing bars will be lessened with greater acceptance and use of the bars. Adaptation of equipment to deal with high-strength reinforcing bars will be a natural consequence of increased use of the bars.

Processes that require changes in the detailing of high-strength reinforcement will require input from the engineering and research communities and an examination of the impacts on current design and construction practices.

2.3 Constructability and Construction Efficiency Issues

The primary benefit of high-strength reinforcement is the potential reduction in the volume of reinforcement needed to accomplish design and construction objectives. As there will likely be a unit cost premium associated with the procurement of high-strength reinforcement, the overall reduction in the volume of reinforcement within a structure will need to be sufficiently large to offset the anticipated cost premium.

A challenge for the design process will involve integration of high-strength reinforcement into concrete structures in ways that optimize and fully utilize the higher yield strength of the bars. Minimum spacing, minimum reinforcement ratios, and other detailing requirements in ACI 318 will influence the ability to reduce the volume of reinforcement, and will have an impact on the relative efficiency of using high-strength reinforcement.

High-strength reinforcement will require longer splice and development lengths than Grade 60 reinforcement. This will negate some of the potential reduction in the volume of reinforcement. The impact of splice and development length issues can be reduced by exploring higher relative rib area designs to increase force transfer to the surrounding concrete. In some cases, the use of higher strength concrete may be necessary to more completely utilize the increased strength of the bars.

If a member cross-section is held constant, substitution of higher grade reinforcement for Grade 60 reinforcement can be implemented by:

- using the same size bars at a larger spacing; or
- using smaller bars at the same spacing.

From a constructability and efficiency perspective, using bars spaced at larger intervals means that construction and cost efficiencies are achieved through lower placement costs, less congestion, and better consolidation of concrete during placement. Fewer bars can also mean fewer lap splices or mechanical splices, but splice lengths may increase unless more transverse reinforcement is provided. Higher strength bars of the same size, however, will be more difficult to bend, couple, and terminate in the field than corresponding Grade 60 reinforcing bars, and rebar cages with fewer bars at a larger spacing are less stable in a free-standing condition during erection.

Using smaller bars at a spacing similar to that required for Grade 60 reinforcement reduces the overall volume of reinforcement, but overall savings in cost and construction efficiencies are reduced. Smaller diameter bars, however, will be easier to bend, couple, and terminate in the field than larger bars of the same grade, and rebar cages with a greater number of bars will be comparatively more stable in a free-standing condition during erection.

Use of high-strength reinforcement in slabs could lead to larger slab deflections. An increase in deflection is a serviceability concern that will impact floor levelness and deflection-sensitive components such as exterior wall enclosures and attachments.

Constructability benefits associated with reduced congestion will be impacted if member cross-sections are reduced to take advantage of reinforcement with higher yield strengths. Designers may be tempted to use smaller member dimensions, in which case congestion will be similar to what occurs in members with Grade 60 reinforcement. Also, the amount of transverse reinforcement may need to be increased (i.e., spacing reduced) to better restrain longitudinal bars from buckling.

The presence of multiple grades of the same size bars on a job site will increase the potential for a contractor to unintentionally install bars of an incorrect grade. This is more likely to occur if the bars are straight and of similar length. Bar markings or other identifiers will need to be made clear to prevent this from occurring. Also, the use of all bar sizes for multiple grades of reinforcement could lead to supply chain, inventory, and inspection complexities for producers, fabricators, and installers. These issues are best addressed early in the design process by focusing on the use of a limited quantity of bar sizes in different grades.

A cost study reported in NIST GCR 14-917-30 determined that savings associated with the substitution of Grade 80 reinforcement for Grade 60 reinforcement was on the order of 4% of the cost of the concrete structure. A study by Price et al. (2014) evaluated the potential for high-strength reinforcement to reduce the volume of reinforcement and construction time in typical concrete structures. This study concluded that if ACI 318 limits on yield strength were ignored, high-strength reinforcement is effective in reducing the volume of reinforcement needed in reinforced concrete building components.

Recommendations. Thorough consideration of constructability issues and cost efficiencies will be necessary for understanding the maximum benefit that can be achieved by using high-strength reinforcement, however, no additional experimental investigation or engineering study is recommended at this time.

Chapter 3

Key Design Issues

This chapter identifies the current state-of-knowledge on key design issues that are affected by the use of high-strength reinforcement. High-strength reinforcement has the potential to impact design provisions throughout ACI 318, as well as other codes and material standards for reinforced concrete construction. Key design issues can be broadly attributed to provisions related to: (1) strength and ductility; (2) serviceability; (3) reinforcement limits; (4) detailing; (5) analysis; and (6) seismic-force-resisting systems.

The use of high-strength reinforcement in design will require demonstration that the mechanical properties of high-strength reinforcement will yield safe and serviceable structures. Resolution of design issues will first require an understanding of the effects of high-strength reinforcement on structural behavior. Once the effects on behavior are established, revisions to design provisions can be considered.

Serviceability considerations are more likely to control the design of members reinforced with high-strength reinforcement than members reinforced with Grade 60 reinforcement. Some serviceability provisions in ACI 318 explicitly considered that higher grade reinforcement (i.e., Grade 80) could be used, while others were based on consideration of Grade 60 reinforcement alone. Use of high-strength reinforcement in members of seismic-force-resisting systems is more likely to take full advantage of higher yield strengths because strength provisions generally control over serviceability concerns in seismic systems. However, minimum spacing, minimum reinforcement ratios, transverse reinforcement requirements, and other detailing provisions in ACI 318 may have the effect of lessening the advantage of using high-strength reinforcement in design.

3.1 Strength and Ductility

Design provisions for computing flexural strength, axial load capacity, and shear strength may need adjustment for application to members reinforced with high-strength reinforcement. Other miscellaneous provisions and approaches to strength computation may also be affected.

3.1.1 Flexural and Axial Load Strength

Strength of reinforced concrete members under flexural, axial, or combined flexural and axial loading is key because design for these load conditions generally establishes the size of the members. Design for other loading effects, such as shear and torsion, necessarily follow. Understanding potential changes in the strength and behavior of members is necessary for confirming the safe and economical use of high-strength reinforcement.

Stress-strain curves for high-strength reinforcement are expected to differ from ASTM A615 Grade 60 reinforcement. Curves representing three different types of stress-strain behaviors for Grade 60, Grade 80, and Grade 100 reinforcing bars are shown in Figure 3-1. Three distinct shapes are possible: (1) a curve defined by three segments (designated S3 in the figure) consisting of linear-elastic behavior to a well-defined yield strength, a relatively flat yield plateau, and a rounded strain-hardening region; (2) a curve defined by two segments (designated S2 in the figure) consisting of linear-elastic behavior to the yield strength, followed by linear strain-hardening behavior until the tensile strength is reached; and (3) a rounded curve (designated S1 in the figure) defined by a gradual reduction in stiffness that becomes nonlinear before reaching a yield strength that is defined by the 0.2% offset method, followed by gradual softening until the tensile strength is reached (also called a "roundhouse" curve).

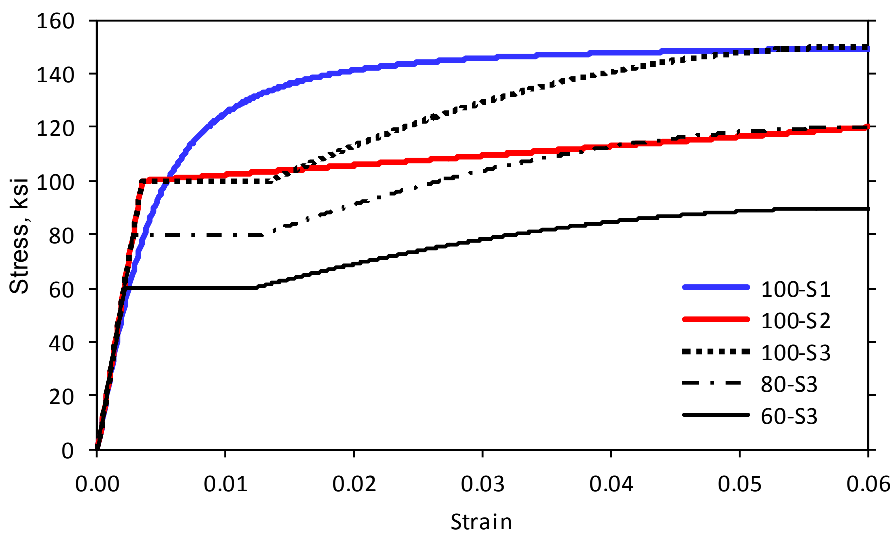


Figure 3-1 Representative stress-strain curve behaviors for Grade 60, Grade 80, and Grade 100 reinforcement (adapted from NIST, 2014).

The stress-strain behavior of ASTM A615 Grade 60 reinforcement is represented by the curve designated 60-S3. Different stress-strain curves are expected to impact the force-displacement behavior of members reinforced with high-strength reinforcement in different ways.

3.1.1.1 Strain Limits

In determining the design strength of a member, it is necessary to reduce the nominal strength through the use of a strength reduction factor, ϕ. The strength reduction factors for flexure and axial load are computed as shown in Figure 3-2, based on the strain conditions of the cross-section at nominal strength, specifically the extreme tensile strain, ε_t. This approach has been in the main body of ACI 318 since 2002, and is based on the *Unified Design Provisions for Reinforced and Prestressed Concrete Flexural and Compression Members* recommended by Mast (1992).

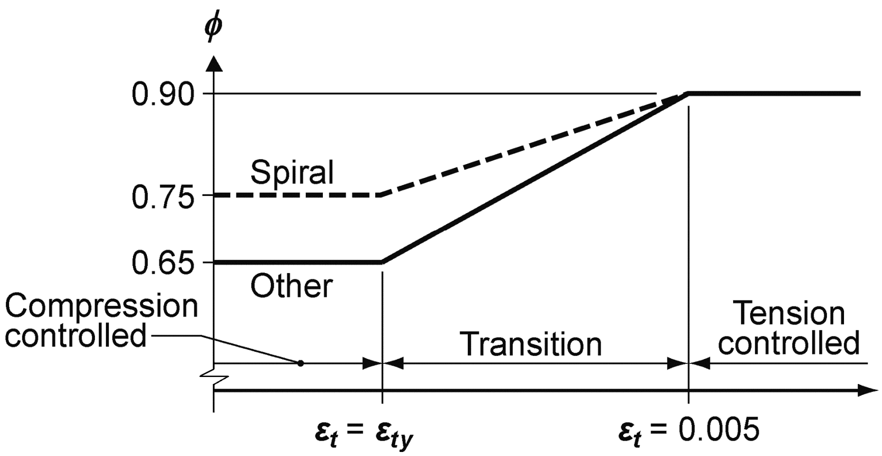


Figure 3-2 Strength reduction factor, ϕ, based on strain (ACI, 2014a).

As shown in Figure 3-2, ϕ depends on the region in which the computed strain lies, and two limits are used to control the value: the compression controlled limit ($\varepsilon_t = \varepsilon_{ty}$), and the tension controlled limit ($\varepsilon_t = 0.005$). In considering the use of high-strength reinforcement, it is important to address whether these two limits remain appropriate.

Compression-Controlled Strain Limit

For the compression-controlled strain limit, ACI 318 defines the yield strain of the reinforcement, ε_{ty}, as f_y/E_s, where f_y is the specified yield strength and E_s is the elastic modulus of the reinforcing steel. Use of the yield strain to define this limit is appropriate regardless of the yield strength of the reinforcement, and computationally correct for reinforcement that remains linear-elastic up to the yield plateau. However, for rounded stress-strain curves with a gradual reduction in stiffness that becomes nonlinear before reaching the yield strength, the definition of the yield strain as f_y/E_s may not be appropriate, especially if f_y is being defined through the 0.2% offset method or other alternative procedure. Therefore, depending on the shape of the stress-strain curve for high-strength reinforcement, changes may be needed in the definition of yield strain. Alternatively, specified values of ε_{ty} for a given reinforcement grade, similar to that provided for prestressed reinforcement, may be required.

Tension-Controlled Strain Limit

For the tension-controlled strain limit, a fixed value of 0.005 is used. This value, which is designed to provide ductility, is approximately 2.5 times the yield strain of about 0.002 for ASTM A615 Grade 60 reinforcement, and is higher than what was required in ACI 318 prior to 2002. According to Mast (1992), the value of 0.005 was selected because:

- the net tensile strain is measured at the distance to the reinforcement closest to the tension face, d_t, instead of the distance to the centroid of tension reinforcement, d, and the limit should be slightly higher when more than one layer of tension reinforcement is used; and
- use of 0.005 produces interaction diagrams that look reasonable.

For high-strength reinforcement, the yield strain will be higher, so it would appear that the tension-controlled strain limit should be increased to provide levels of ductility consistent with ASTM A615 Grade 60 reinforcement. As a result, it may be appropriate to provide a tension-controlled strain limit that is a function of the yield strain in the reinforcement.

As in the case of the compression-controlled strain limit, the use of f_y/E_s to define the yield strain may not always be appropriate, and the shape of the stress-strain curve may affect the strain that should be used for the tension-controlled limit. For example, ACI ITG-6R, *Design Guide for the Use of ASTM A1035/A1035M Grade 100 Steel Bars for Structural Concrete* (ACI, 2010a), notes that the tensile strain limit for ASTM A1035 reinforcement is 0.0066, based on realistic stress-strain curves, and 0.009, based on a simplified representation of the highly nonlinear ASTM A1035 stress-strain curve idealized as elastic-perfectly-plastic.

Maximum Strain Limit

There is likely to be a reduction in the elongation capacity of high-strength reinforcement relative to Grade 60 reinforcement. Although ACI 318 does not currently include a maximum limit on tensile strain, it is possible that a limit will need to be established so that fracture of the reinforcement does not occur prior to crushing of the concrete in compression.

Recommendations. An analytical study as described in Section 4.4.1 should be performed to investigate strain limits defining the boundaries between compression-controlled, transition, and tension-controlled sections with high-strength reinforcement. In addition, the study should investigate the need for a maximum strain limit for avoiding fracture prior to concrete crushing in sections with high-strength reinforcement.

3.1.1.2 Beams

Flexure strength provisions for tension-controlled members, which include most slabs and beams, are intended to provide an acceptable level of reliability that the computed design strength will be achieved at a reasonable deflection. They are also intended to result in designs that provide warning of potential failure if a member is inadvertently overloaded. Warning signs include substantial cracking and large deflections prior to failure or collapse. These provisions were developed considering properties of reinforcing steel that included a yield plateau, and adequate performance was confirmed through experimental tests of beam specimens in laboratories.

Evaluation of current flexural strength design provisions for high-strength reinforcement should consider the following issues:

- The shape of the stress-strain curve of the reinforcing steel could affect the deflection at which the design strength is realized, as well as the spread of plasticity as the member is loaded monotonically to failure.
- A minimum extreme tensile strain ε_t of 0.004 is required by ACI 318 for flexural members.
- The yield strain of compression reinforcement will exceed the assumed maximum concrete strain of 0.003.

Stress-Strain Curve

The load-deflection behavior of tension-controlled members reinforced with high-strength reinforcement will be affected by the shape of the stress-strain curve of the reinforcing steel. Current provisions for computing flexural strength are based on the assumption that the stress-strain curve includes a yield plateau. The effects of the shape of the stress-strain curve should be investigated to determine whether or not changes to current flexural design provisions are required. This could be accomplished using moment-curvature studies coupled with experimental tests to validate the analyses.

The desire for large deflections prior to flexural failure is considered in ACI 318. The ability to develop large deflections depends, in part, on the mechanical properties of the reinforcement. Reinforcing bars with a yield plateau and a large tensile-to-yield strength ratio will result in members that can deflect more before failure occurs. For ASTM A615 Grade 60 reinforcing bars currently being produced, about 95% have a yield plateau (Paulson et al., 2013). A statistical analysis of the mechanical properties of reinforcing bars by Bournonville et al. (2004) indicates that the average tensile-to-yield strength ratio is about 1.5 for most bars, and about 1.4 for larger bars. Although ASTM A615 does not specify a minimum acceptable value for tensile-to-yield strength ratio, most reinforcing bars currently being produced have acceptable values for this ratio in most cases. Experimental testing should be performed to confirm that slabs and beams reinforced with high-strength reinforcement (i.e., Grade 100) will deflect sufficiently to provide adequate warning prior to failure.

Minimum Tensile Strain

According to ACI 318, a minimum tensile strain is required to ensure a minimum level of ductility. The current limit of 0.004 is approximately twice the yield strain of about 0.002 for ASTM A615 Grade 60 reinforcement. Although this limit was not part of the recommendations in Mast (1992), it was included in ACI 318-02 to provide consistency with past practice, which included a maximum longitudinal reinforcement ratio of $0.75\rho_b$, which is the ratio of steel to concrete area producing a balanced strain condition. The maximum reinforcement ratio produces a net tensile strain, ε_{ty}, of 0.00376 in Grade 60 reinforcement, which is the reason ACI 318 adopted the slightly more conservative value of 0.004, but with a penalty on the strength reduction factor. Considering that this ratio was selected based on Grade 60 reinforcement, an adjustment may be required for high-strength reinforcement. Another possibility would be to delete this requirement and address only the tension-controlled limit, as was originally recommended by Mast (1992).

Compression Reinforcement

Compression reinforcement is commonly used in beams. In some cases, compression reinforcement is provided in beams with smaller cross-sections to allow the use of more tension reinforcement. For high-strength reinforcement in compression, the yield strain is likely greater than the assumed maximum concrete strain of 0.003. As a result, the design stress in high-strength compression reinforcement will be limited by the strain profile of the cross-section, or by a prescribed limit in the maximum specified yield strength, such the current 80 ksi limit in ACI 318. Considering that a strain of 0.003 would result in a stress of 87 ksi in the reinforcing steel, it would seem that no change in ACI 318 is needed to address this issue. However, because the stress in the compression reinforcement will be limited by the assumed maximum concrete compressive strain, it appears that the current 80 ksi limit on the yield strength of compression reinforcement could be removed without complication.

Recommendations. An analytical study as described in Section 4.4.1 should be performed to evaluate the impact of the shape of different stress-strain curves on the flexural strength and load-deflection behavior of flexural members. Computed flexural strengths should be compared with code-predicted nominal strengths to determine if changes are needed to traditional ACI 318 design assumptions for calculating flexural strength (i.e., equivalent stress block for concrete and elastic-plastic stress-strain curve for reinforcing steel).

In addition, experimental testing as described in Section 4.4.2 should be performed to determine the load-deflection behavior of beams reinforced with high-strength (i.e., Grade 100) reinforcement. Tests should consider the actual tensile-to-yield strength ratios and elongations that are likely to be achieved in the production of high-strength reinforcement.

It is possible that the 80 ksi limit on yield strength for compression reinforcement in ACI 318 could be removed, but such a change is not considered necessary.

3.1.1.3 Columns

The yield strength of compression reinforcement is limited to 80 ksi in ACI 318. This limit is imposed because bars with yield strengths much above 80 ksi will not contribute to increased compression capacity because higher yield strengths can only be achieved at yield strains above 0.003, which is the strain assumed for crushing of concrete. At an assumed maximum strain of 0.003, the maximum usable stress in the reinforcing steel would be 87 ksi, considering linear-elastic behavior.

Richart and Brown (1934) reported the results of a test program in which 564 concentrically loaded columns were tested. Eight of the tests were spirally reinforced columns with longitudinal reinforcing bars having a yield strength of 96 ksi, tensile strength of 133 ksi, and fracture elongation of 11% (8-inch gauge length). The stress-strain curves for the longitudinal bars had well-defined yield plateaus. The columns had 4% longitudinal reinforcement and 1.2% spiral reinforcement (volumetric reinforcement ratio) spaced at one-sixth of the core diameter, and the spiral reinforcement had a yield strength of 64 ksi and tensile strength of 100 ksi. The report concluded that the high-strength longitudinal bars "were fully effective in producing the column strength."

Pfister and Mattock (1963) reported the results of 16 concentrically loaded circular and rectangular columns reinforced with high-strength reinforcing bars both with and without splices. The 12-inch diameter circular columns were spirally reinforced, and 10-inch by 12-inch rectangular columns had ties. The longitudinal reinforcement consisted of six No. 8 bars, for a reinforcement ratio of about 4%, with yield strengths between 82 ksi and 93 ksi. The yield strength of the transverse reinforcement was 65 ksi for the spirals and 59 ksi for the ties. The volumetric ratio of the spiral reinforcement was 1.3%, while the area ratio of the ties was 0.1% (very low, but compliant with the minimum permitted in ACI 318-63). The report concluded, "If the specified yield point of longitudinal reinforcement in tied columns is to be developed at ultimate strength of the columns, then it is necessary that the yield point be reached at or before a strain of 0.003 inches/inch. This condition will normally be more readily complied with by bars having a clearly defined yield point and a nearly linear stress-strain curve up to yield than by bars having a gradually curving stress-strain curve with no clearly defined yield point." The report also stated, "In future usage of 90 ksi column reinforcement, lapped splices will probably be impractical even with spiral reinforcement."

Todeschini et al. (1964) tested eccentrically loaded tied columns with longitudinal reinforcing bars with a specified yield strength of 75 ksi. Reinforcement both with and without yield plateaus were used. Reinforcement with a rounded stress-strain curve shape reached 90 ksi at a strain of 0.006. The report concluded that the shape of the stress-strain curve had an effect on the stress that could be realized in the reinforcement. For reinforcement with a rounded stress-strain curve, stresses on the order of 70 ksi to 80 ksi could be developed. For reinforcement with a relatively flat yield plateau, stresses up to 90 ksi could be developed.

More recent test programs and findings are presented in NIST GCR 14-917-30, *Use of High-Strength Reinforcement in Earthquake-Resistant Concrete Structures* (NIST, 2014). For members reinforced with high-strength reinforcement subjected to reverse cyclic loading, test results indicate that replacing Grade 60 longitudinal reinforcement with reduced amounts of high-strength reinforcement (reduced in proportion to the yield strength of reinforcement) leads to comparable flexural strength and deformation capacity. Specimens in these tests included reinforcement with limited variation in the shape of the stress-strain curve.

It should be understood that maximum stresses in compression reinforcement will only be realized under pure axial load conditions. Strain gradients will decrease the strain (and the stress) in the compression reinforcement, and most practical columns include an interaction between axial load and flexure (*P-M* interaction). Overall, the computation of the *P-M* interaction curve is controlled by the maximum concrete strain and the strain gradient. Because high-strength reinforcement yields at higher strains, the strain gradient will be affected as well as the shape of the *P-M* interaction curve around the balance point. As noted in Section 3.1.1.1, selection of the compression-controlled and tension-controlled strain limits will be important, as these will significantly impact the design strength curves.

Recommendations. *P-M* interaction for columns should be considered as part of analytical studies evaluating compression-controlled and tension-controlled strain limits described in Section 4.4.1.

Past experimental testing has shown that the shape of the stress-strain curve can affect the stress developed in longitudinal column reinforcement. An analytical study as described in Section 4.4.3 should be performed to evaluate the impact of the shape of different stress-strain curves on column strength, and to investigate the possible elimination of the 80 ksi limit on yield strength for compression reinforcement in columns. Such a change would result in the 0.003 strain limit for concrete controlling the design, with the stress in the reinforcement computed from the stress-strain relationship.

3.1.2 Shear Strength

An overarching design philosophy is to provide shear strength that exceeds the shear demand corresponding to the flexural strength, which is intended to allow for more complete flexural yielding and warning prior to failure. In ACI 318, shear strength consists of the addition of a concrete component, V_c, and a shear reinforcement component, V_s. This approach is used throughout ACI 318, and is provided for both one-way and two-way shear strengths. Although the most commonly used shear design expressions are based on the assumption that shear strength is independent of flexure, research has shown that flexural reinforcement significantly affects shear strength. Consequently, the effects of high-strength flexural reinforcement on shear strength need to be considered along with the direct effects of using of high-strength reinforcement as shear reinforcement.

3.1.2.1 Shear Strength of Members without Shear Reinforcement

Some members (e.g., slabs and foundations) are often designed without the use of shear reinforcement. Depending on size and geometry, beams may not require shear reinforcement in certain regions of the span (or in any part of the span). The use of high-strength reinforcement as flexural reinforcement may affect the concrete contribution to shear strength. In cases where high-strength reinforcement is used to reduce the amount of longitudinal reinforcement, higher strains are likely to occur over the cross-section for the same loading. Higher longitudinal strains will result in larger crack widths, which have the potential to reduce shear transferred across the cracks (Angelakos et al., 2001; Brown et al., 2006; Collins and Kuchma, 1999; Lubell et al., 2004; Reineck et al., 2003; Tureyen and Frosch, 2003). In addition, a reduction in the amount of longitudinal reinforcement reduces the depth of the compression zone which is considered a primary region contributing to shear transfer. Overall, it is important to determine if adjustments in commonly used shear expressions are necessary.

3.1.2.1.1 Beams, One-Way Slabs, and One-Way Foundations

The common expression used to compute one-way shear strength of concrete is $2\sqrt{f_c'}b_w d$, where f_c' is the specified concrete compressive strength, b_w is the web width, and d is the depth to the centroid of the longitudinal tension reinforcement. This expression is independent of the flexural reinforcement, and was developed considering the lower-bound of test results available in the 1960s (ACI-ASCE Committee 326, 1962). Although this expression has been in use for decades, interaction between shear and flexure has been known for some time, and ACI 318 provides an alternative equation that includes this interaction.

Research by Tureyen and Frosch (2003) clearly illustrates the influence of the reinforcement ratio on the shear strength of concrete. In evaluating shear tests available in the literature along with more recent tests, the significant influence of the reinforcement ratio, and thus the interaction between flexure and shear, can be seen in Figure 3-3. As the reinforcement ratio decreases below 1%, shear strengths below $2\sqrt{f_c'}$ are observed. In a research study by Tureyen and Frosch (2002), a specimen was reinforced with high-strength reinforcing bars with a yield strength of 108 ksi. It was found that the behavior of this specimen was essentially identical to that of a companion fiber-reinforced polymer (FRP) reinforced specimen for the same effective reinforcement ratio. For the same axial stiffness of the reinforcement, the same overall behavior occurred and the same shear strengths developed. Therefore, if the reinforcement ratio is maintained constant between different reinforcement grades, a reduction in shear strength is not expected, and the use of high-strength reinforcement alone is not expected to change overall shear behavior. It is important to note that shear tests are designed such that the longitudinal reinforcement does not yield, and maintaining the longitudinal reinforcement in the elastic region is fundamental in clearly identifying shear failures and shear strength.

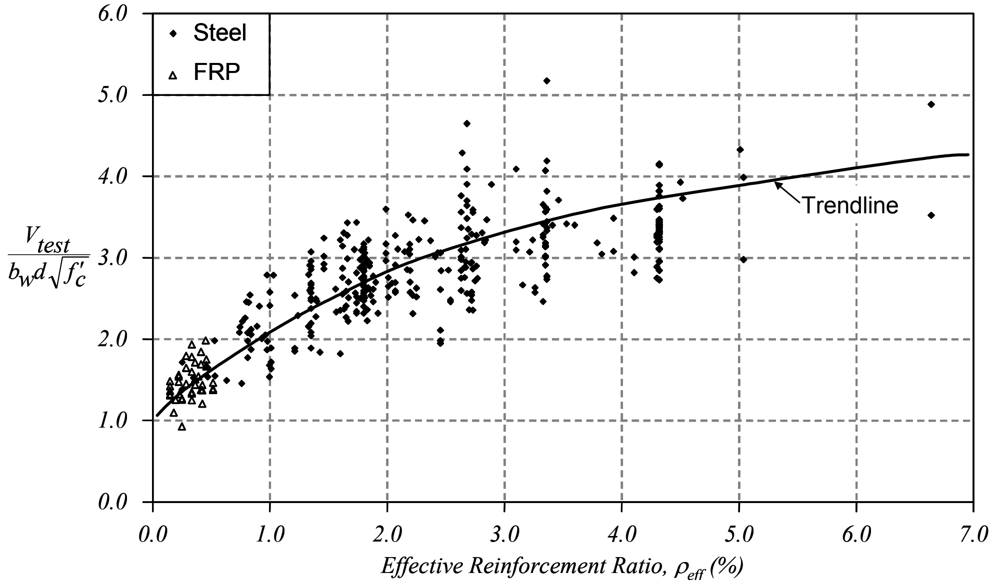


Figure 3-3 Influence of reinforcement ratio on shear strength (Tureyen and Frosch, 2003).

Although shear behavior is not expected to change with the use of high-strength reinforcement, it is expected that the use of high-strength reinforcement will reduce the required amount of longitudinal reinforcement, which will decrease shear strength. Furthermore, it is expected that reinforcement ratios below 1% will be common. Therefore, a change in the expression used to calculate the concrete contribution to shear strength is likely needed.

The expression proposed by Tureyen and Frosch (2003), $5\sqrt{f_c'}b_w c$, where c is the distance between the extreme compression fiber and the neutral axis, can potentially provide a solution. This expression has been adopted for the design of FRP-reinforced members in ACI 440.1R-06, *Guide for the Design and Construction of Structural Concrete Reinforced with FRP Bars* (ACI, 2006) for the reasons illustrated in Figure 3-3. FRP-reinforced structures have low effective reinforcement ratios (i.e., less than 1%) due to the relatively low modulus of elasticity of these materials (often 1/4 to 1/5 of the value for steel), and a different expression for shear strength was required.

Recommendations. Experimental testing as described in Section 4.4.5.1 should be performed to evaluate current design expressions for the shear strength of one-way members with high-strength longitudinal reinforcement, and determine an appropriate expression for the calculation of the concrete contribution to shear strength, V_c.

3.1.2.1.2 Two-Way Slabs and Two-Way Foundations

In two-way slabs and foundations, punching shear strength typically controls the design. Punching shear strength has many of the same issues outlined for one-way shear strength. In ACI 318, the basic expression for punching shear strength is $4\sqrt{f_c'}b_o d$, where f_c' is the specified concrete compressive strength, b_o is the perimeter of the critical section in shear, and d is the depth to the centroid of the longitudinal tension reinforcement. As in the case of one-way shear, experimental results indicate that the longitudinal reinforcement ratio has a significant influence on shear strength. Ospina (2005) evaluated punching shear strength from tests available in the literature. Results are shown in Figure 3-4, indicating that shear capacity ratios (V_{test}/V_{calc}) below 1.0 result as the longitudinal reinforcement ratio is decreased.

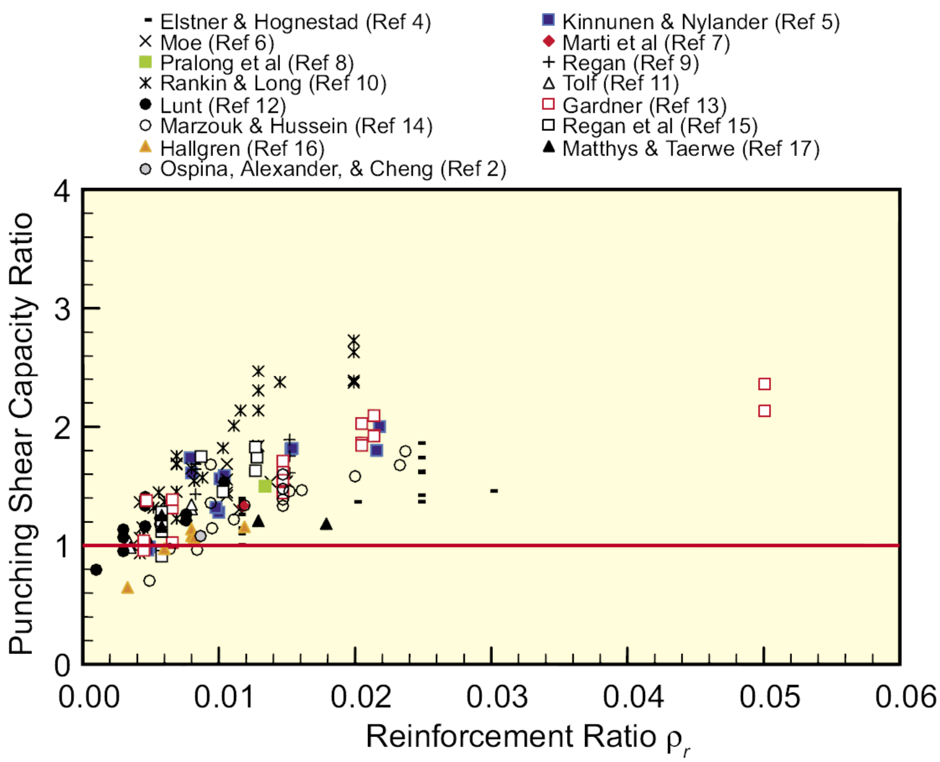


Figure 3-4 Ratios of tested to predicted punching shear capacity using ACI 318-05 (Ospina, 2005).

With the use of high-strength reinforcement, it is expected that lower reinforcement ratios would be used. Therefore, a design approach that accounts for the reduction in reinforcement ratio may be needed. Ospina (2005) recommended $10\sqrt{f_c'}b_o c$, which is a modification of the Tureyen and Frosch expression used for one-way shear. This approach has been adopted for two-way shear in FRP-reinforced members in ACI 440.1R-06, and may also be appropriate for members with high-strength longitudinal reinforcement.

Recommendations. Experimental testing as described in Section 4.4.5.2 should be performed to evaluate current design expressions for the punching shear strength of two-way members with high-strength longitudinal reinforcement, and determine an appropriate expression for the calculation of the concrete contribution to shear strength, V_c.

3.1.2.2 Shear Strength of Members with Shear Reinforcement

The shear strength of members with shear reinforcement is computed through the addition of the concrete contribution to shear strength, V_c, and the contribution from shear reinforcement, V_s. The concrete contribution shear strength is identical to that for members without shear reinforcement, and is based on tests results. The contribution from shear reinforcement is based on the strength of an individual stirrup multiplied by the number of stirrups assumed to cross an inclined shear crack. The strength of a stirrup is taken as the area of the legs of the stirrup multiplied by the yield strength of the stirrup. Although tests (ACI-ASCE Committee 326, 1962; ACI-ASCE Committee 426, 1973) support the use of yield strength, these tests were focused on yield strengths up to 60 ksi. Therefore, the ability to fully utilize the yield strength of high-strength shear reinforcement should be investigated.

ACI 318 currently limits the design yield strength of shear reinforcement to 60 ksi for reinforcing bars and 80 ksi for welded deformed wire reinforcement. The commentary states that the limit of 60 ksi "provides a control on diagonal crack width," and the higher yield strength of 80 ksi can be used for deformed wire reinforcement because "the widths of inclined shear cracks at service load levels were less for beams reinforced with smaller-diameter welded deformed wire reinforcement cages designed on the basis of a yield strength of 75,000 psi than beams reinforced with deformed Grade 60 stirrups." Considering that the ACI 318 limit on yield strength is based on serviceability considerations rather than strength, it may not be possible to fully utilize the yield strength of high-strength shear reinforcement.

Overall, there are two primary issues to be addressed in considering the use of high-strength reinforcement as shear reinforcement: (1) yield strength in the shear reinforcement that can be utilized in the development of shear strength; and (2) crack control when stresses in the transverse reinforcement exceed 60 ksi. It is also important to verify that the traditional formulation of the concrete contribution, V_c, plus the shear reinforcement contribution, V_s, remains applicable.

Several studies have been conducted investigating the use of high-strength reinforcement as shear reinforcement (Aoyama, 2001; Budek et al., 2002; Munikrishna, 2008; Ou et al., 2012; and Sumpter et al., 2009). In addition, a National Cooperative Highway Research Program (NCHRP) research project was conducted to evaluate the use of high-strength reinforcement (Shahrooz et al., 2011) and tests were included specifically evaluating the use of high-strength reinforcement as shear reinforcement. In general, these studies indicated that an increase above the current 60 ksi limit for shear reinforcement is possible. In the NCHRP study, only small differences in crack widths between Grade 60 and Grade 100 stirrups were observed, indicating that crack control may be possible at higher stress levels.

As a secondary issue, ACI 318 requires a minimum area of shear reinforcement. This requirement is currently based on the prevention of sudden shear failures when cracking occurs, and is a function of $\sqrt{f_c'}$. However, the total force provided, $A_{v,\min} f_{vt}$, is based on the area and the yield strength of the transverse reinforcement. Therefore, if the transverse reinforcement is not capable of reaching yield, the requirement for minimum area of shear reinforcement requirement may need modification.

Recommendations. Experimental testing as described in Section 4.4.6 should be performed to evaluate current design expressions for the concrete contribution, V_c, plus the shear reinforcement contribution, V_s, to shear strength for flexural members incorporating both longitudinal and transverse high-strength reinforcement. This study should also investigate service level crack widths for shear reinforcement greater than 60 ksi, as well as the requirement for minimum area of shear reinforcement.

3.1.2.3 Shear-Friction

Design for shear forces that are transferred across a shear plane, such as an existing or potential crack, an interface between dissimilar materials, or an interface between two concrete surfaces that were cast at different times, uses the shear-friction design method in which shear strength is calculated as the normal force across the plane multiplied by a coefficient of friction, μ. As shear is applied across the interface, the surfaces are assumed to slip (or slide) along each other. Irregularities in the surfaces cause the horizontal slip to be accompanied by vertical separation between the surfaces. This vertical separation produces strains (and stresses) in the reinforcement crossing the interface. At peak strength, the separation is sufficient to yield the reinforcement, and the normal force is computed as the product of the area of the reinforcement and the yield strength of the reinforcement, $A_{vf} f_y$. The coefficient of friction is different depending on the condition (i.e., roughness) and the materials of each surface at the shear plane.

Considering a shear plane with a given amount of reinforcement crossing the interface, the lowest force is transferred across the interface when concrete is placed against hardened concrete that is not intentionally roughed. In this case, μ is assumed to be 0.6, vertical separation is negligible due to the smooth interface, and the shear reinforcement is acting primarily as a dowel. The yield strength of a dowel is computed as approximately $0.6f_y$ which is consistent with the commonly assumed shear yield strength of steel.

Considering the same shear plane and amount of reinforcement crossing the interface, the greatest force is transferred when concrete is placed monolithically. In this case, μ is assumed to be 1.4 because a jagged and irregular interface will be formed when the concrete cracks along the interface. At this roughness, maximum vertical separation will occur, and shear strength is provided by a combination of shear yielding of the reinforcement ($0.6f_y$) and friction along the interface caused by the normal force. In a testing program by Valluvan (1993), specimens were constructed with a vertical force applied across a shear plane, but without reinforcement across the plane. In these tests, a coefficient of friction of 0.8 was observed. Similar results were obtained by Mattock and Hawkins (1972). As a result, the alternative shear friction equation presented in the commentary to ACI 318-11 uses a coefficient of friction of 0.8, and the shear strength provided by the normal force may be considered to be $0.8f_y$. The addition of the shear yielding component with the normal force component produces a total of $1.4 f_y$, which is consistent with the ACI 318 value for the coefficient of friction.

ACI 318 currently limits the yield strength used for the design of shear-friction reinforcement to 60 ksi. This limit is primarily based on a lack of available test data using higher strength reinforcement. Therefore, it is not known if adequate separation will develop to induce sufficient strains to develop the yield strength of high-strength reinforcement. For the lower-bound strength of $0.6f_y$ for concrete placed against hardened concrete that is not intentionally roughened, it would appear that modification is not needed because this value is controlled by the shear strength of the reinforcement and full yield strength will not be utilized. However, further investigation will be needed to determine if the full yield strength of high-strength reinforcement can be realized in producing normal forces across a shear plane.

Recommendations. Experimental testing as described in Section 4.4.7 should be performed to investigate the shear strength along an interface reinforced with high-strength shear-friction reinforcement.

3.1.2.4 Deep Beams

Deep beams are given special consideration in ACI 318 due to their span-to-depth ratio, which results in the member being a region of discontinuity where plane-sections do not remain plane. There are two approaches for design of deep beams: (1) nonlinear finite element analysis which accounts for the nonlinear distribution of longitudinal strain over the depth of the beam; or (2) strut-and-tie models.

In general, a deep beam acts as a tied arch with the longitudinal reinforcement acting as a tension tie. Based on this mechanism, higher strength reinforcement should be effective, assuming that the reinforcement is properly anchored. Several of the provisions for deep beams, however, are focused on serviceability considerations such as crack control. For example, the shear stress in deep beams is limited to $10\sqrt{f_c'}$, which is used to impose a dimensional restriction that controls cracking under service loads while also protecting against diagonal compression failures at ultimate strength. Also, distributed reinforcement is required along the side faces, and a minimum reinforcement ratio of 0.25% is required both perpendicular and parallel to the axis of the beam.

Birrcher et al. (2013) conducted experimental tests to evaluate the minimum web reinforcement requirement in deep beams. Twelve full-scale tests were conducted in which the shear span-depth ratios (a/d) were 1.2, 1.85, and 2.5. At each value of a/d, the quantity of web reinforcement was the primary variable. Web reinforcement ratios ranged from 0 to 0.3% in the vertical and horizontal directions, and the concrete compressive strength ranged from 3,200 psi to 5,000 psi. Results indicated that a larger quantity of web reinforcement was needed to adequately restrain the width of diagonal cracks than was necessary for providing adequate deep beam shear capacity. Based on these strength and serviceability results, a minimum web reinforcement ratio of 0.3% in each orthogonal direction was recommended for deep beams, which is slightly higher than ACI 318 requirements. Although Grade 60 reinforcement was used in the testing program, it is not expected that higher strength reinforcement would impact this minimum reinforcement requirement, and it does not appear that modification of deep beam provisions will be needed for the use of high-strength reinforcement.

Recommendations. No further study is recommended or required at this time, and a code change is not considered necessary.

It should be noted that development and anchorage of high-strength reinforcement is discussed in Section 3.4.4, and additional investigation is needed in this area. In the case of tension reinforcement in deep beams, the short distance between peak tension and the end of the shear span may require special anchorages for high strength reinforcing bars.

3.1.2.5 Ordinary Structural Walls

Structural walls are commonly used for gravity support and lateral-force resistance. Ordinary structural walls are acceptable for wind loading, and for earthquake loading in regions of low or moderate seismicity. In the case of wind loads, elastic response is intended. For earthquake loads, inelastic response is anticipated. Design in accordance with ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010), includes a response modification factor, R, which intentionally reduces earthquake design forces below the level required for elastic behavior. The value of R is taken as 4 if the walls are part of a bearing wall system, and 5 if the walls are part of a frame system.

The behavior and design of ordinary structural walls depends on the aspect ratio of the wall. Tall, slender walls are flexure-dominated, and short, squat walls are shear-dominated. The design of horizontal wall reinforcement is typically controlled by the required strength to resist in-plane shear forces. For slender walls, vertical reinforcement of ordinary walls is typically controlled by the design for combined axial load and in-plane flexure. For squat walls, the required vertical reinforcement can be controlled by: (1) the combined axial load and in-plane flexure; or (2) requirements for vertical reinforcement to match (or be a function of) the horizontal reinforcement. The combined axial and flexural strength is computed assuming plane sections remain plane, with the concrete compression strain at the extreme fiber equal to 0.003. For walls with aspect ratios less than 2.5, minimum requirements for vertical reinforcement relative to horizontal reinforcement reflect the participation of vertical reinforcement in shear resistance.

In ACI 318, nominal shear strength calculations in ordinary structural walls include both concrete and reinforcement contributions to shear resistance, with a combined limit equal to $10\sqrt{f_c'}hd$, where h is the wall thickness, and d is the flexural depth, often taken as 80% of the wall length. The concrete contribution to shear strength, V_c, can be computed using the simplified formula of $2\sqrt{f_c'}hd$ or the more detailed formula in Table 11.5.4.6 of ACI 318-14, which includes the interactive effects of shear with both axial and flexural forces.

Ordinary structural walls are commonly used in tall buildings with height-to-length aspect ratios typically greater than 3:1. Walls with aspect ratios of three and greater are considered slender, and are expected to behave in accordance with beam theory. Although most ordinary structural walls are 12 inches to 18 inches thick, some are as much as 30 inches thick, reinforced with No. 18 and larger high-strength reinforcing bars, with concrete compressive strengths of up to 15,000 psi. Although ordinary slender structural walls may experience nonlinear response, the ends of the walls are not required to have transverse confining reinforcement (which is used to provide stable post-yield concrete compressive strength and buckling restraint for longitudinal reinforcement in special structural walls).

Issues with the use of high-strength reinforcement in ordinary structural walls include the following:

- ACI 318-14 limits the yield stress used for the design of shear reinforcement to 60 ksi. This is intended to control service level cracking.
- Grade 100 and Grade 120 reinforcement yields in compression at strains exceeding 0.003, which is the strain at which concrete is assumed to crush.
- Higher bond stresses for high-strength reinforcement are more likely to split the cover concrete at splices.
- The behavior of walls with No. 18 and larger bars with high-strength concrete may be different than walls with smaller bars and conventional strength concrete.

Recommendations. Experimental testing as described in Section 4.9.7 and Section 4.9.8 should be performed on ordinary structural walls with high-strength reinforcement.

3.1.3 Strut-and-Tie Modeling

ACI 318 allows for members to be designed using strut-and-tie models. Although the approach is available for members in general, it is most commonly used in the design of discontinuous regions, also known as D-regions. Strut-and-tie modeling is a plasticity approach; therefore, it provides a safe solution for the capacity of the structure. It does not, however, address serviceability issues such as deflections or cracking (Schlaich et al., 1987). Deflections are typically estimated using elastic analyses of the strut-and-tie system. Crack widths are typically controlled as follows: (1) crack widths in tie elements are controlled using the crack-control provisions for flexural members; and (2) crack widths within strut elements are controlled through distributed reinforcement (e.g., 0.3% reinforcement ratio) provided across the strut.

Because strut-and-tie modeling is a plasticity approach, its use remains theoretically applicable for high-strength reinforcement. However, high-strength reinforcement must have adequate ductility and anchorage. Also, with the use of higher strength reinforcement, it is expected that deflections and crack widths will increase, so serviceability issues must be addressed in design.

Recommendations. Assuming that adequate reinforcement ductility and anchorage are provided, no further study of strut-and-tie modeling is recommended or required at this time, and a code change is not considered necessary.

3.1.4 Other Strength Issues

Several other miscellaneous issues related to the strength of members should be considered before adopting high-strength reinforcement into ACI 318. These include establishing appropriate concrete strengths to use, whether or not higher yield strengths can be used in tension regions of shells and folded plates, and whether or not higher yield strengths can be used when computing required quantities of bonded reinforcement for members constructed with unbonded post-tensioning tendons.

3.1.4.1 Concrete Strength

Existing literature includes tests combining high-strength concrete with high-strength reinforcement (e.g., Aoyama, 2001; Okamoto et al., 2004; Nishiyama, 2009; Restrepo et al., 2006). It is considered advantageous to use high-strength concrete in members that will use high-strength reinforcement. High concrete strength will reduce the required development and splice lengths of reinforcement, improve deformation capacity of flexural members, increase the shear strength of members, improve the strength of columns with high axial loads or combined axial load and flexure, increase the shear strength of joints in special moment frames, and reduce deflections.

Use of high-strength concrete improves the deformation capacity of flexural members. Considering two beams close to their ultimate deformation capacity, the beam with higher strength concrete will have a shallower neutral axis depth, slightly increased peak moment strength, more curvature, higher bar tensile strain, and larger hinge rotation.

Similarly, high-strength concrete improves the strength of columns with combined axial load and flexure because a smaller stress block can support the same compressive force. For the same axial load, a smaller stress block will increase the distance between the tension and compression couple, resulting in higher flexural strength.

Raising the minimum concrete compressive strengths in Section 19.2.1.1 of ACI 318 (e.g., 2,500 psi for members except those of special moment frames and special shear walls, which require a minimum concrete compressive strength of 3,000 psi) is not considered necessary for members using high-strength reinforcement, however, it may not be practical to design and construct such members without higher strength concrete.

Specific investigation of the use of high-strength concrete in members reinforced with high-strength reinforcement is beyond the intended scope of this *Roadmap*. However, some testing with concrete compressive strengths on the order of 8,000 psi to 15,000 psi are recommended in selected tests throughout this *Roadmap*, as concrete strengths this high are becoming more common, and are likely to be used even more frequently in the future.

Recommendations. Experimental testing specifically targeted to explore the use of high-strength concrete in combination with high-strength reinforcement is not recommended at this time. However, variation in concrete compressive strength is one of many variables considered in the tests outlined in this *Roadmap*.

3.1.4.2 Tension Regions of Shells and Folded Plates

Shells and folded plates are configured to take advantage of the concrete acting in compression, so tension regions are often minimized. An example of a tension region is the circumferential direction in the portion of a dome that is below an angle of 52 degrees measured from vertical, and an example of a tension element is a ring beam at the base of the dome.

Use of high-strength reinforcement will reduce the required area of reinforcement in tension regions, which will increase the strain in the reinforcement, increase the level of concrete cracking, and reduce the stiffness. The effect of reduced stiffness, and whether reduction in stiffness is acceptable from a structural standpoint, requires investigation. In addition, shells and folded plates are often used as roof elements without an additional roof membrane, so increased cracking could lead to serviceability and durability concerns if the cracks are significantly wider.

Recommendations. Structural and serviceability concerns should be investigated with a combined experimental and analytical study as described in Section 4.4.4. The level of stiffness reduction and magnitude of cracking should be assessed with tests on panels reinforced with high-strength reinforcement, and finite element analyses should be used to evaluate the effects of reduced stiffness.

3.1.4.3 Bonded Reinforcement Ratios for Members with Unbonded Post-Tensioning

A minimum amount of bonded reinforcement is required in ACI 318 for prestressed beams with unbonded tendons. According to the commentary, this reinforcement is provided "to ensure flexural behavior at ultimate beam strength, rather than tied arch behavior, and to limit crack width and spacing at service load when concrete tensile stresses exceed the modulus of rupture." The commentary further states that "Providing minimum bonded reinforcement helps to ensure acceptable behavior at all loading stages." The minimum requirement for bonded reinforcement is based on research by Mattock et al. (1971) that compared the behavior of bonded and unbonded post-tensioned beams, and is independent of the yield strength of the reinforcement. Considering the basis, it does not seem necessary to alter the minimum requirement for bonded reinforcement when high-strength reinforcement is used.

Recommendations. No further study is recommended or required at this time, and a code change is not considered necessary.

3.1.4.4 High-Cycle Elastic Fatigue of Reinforcement

There is a potential for reduced high-cycle elastic fatigue resistance of high-strength reinforcement relative to Grade 60 reinforcement. ACI 215R, *Considerations for Design of Concrete Structures Subjected to Fatigue Loading* (ACI, 1997), addresses fatigue of reinforced-concrete members. Although fatigue resistance is generally not a concern for building structures, fatigue is an important material issue for bridges and, possibly, selected non-building structures. Testing for fatigue resistance is not needed for adoption of high-strength reinforcement into ACI 318 but would be needed for adoption into the American Association of State Highway and Transportation Officials (AASHTO) *LRFD Bridge Design Specifications* (AASHTO, 2014). Research that supports the AASHTO design provisions for elastic fatigue of steel reinforcing bars is reported in National Cooperative Highway Research Program (NCHRP) Report 164, *Fatigue Strength of High-Yield Reinforcing Bars* (Helgason et al., 1976). For high-strength reinforcing bars of Grade 100 and Grade 120, fatigue testing will likely be required for bars manufactured by micro-alloying, quenching, and self-tempering. It could also be required for different bar deformation arrangements. For example, higher relative rib area bars may be more susceptible to fatigue (although no problems were observed in Grade 60 high relative rib area bars reported by Fei and Darwin, 1999). High-cycle elastic fatigue resistance is likely a function of the radius at the base of the deformation and potential defects at this location. Fatigue of mechanical splices was studied as part of NCHRP Project 10-35 (Paulson and Hanson, 1991), and mechanical splicing of high-strength reinforcing bars might also need investigation.

Testing for high-cycle fatigue resistance of reinforced-concrete members is more likely to be a problem in bridge structures with sponsorship provided by the Transportation Research Board (TRB) of the National Academies and the Federal Highway Administration (FHWA) through the National Cooperative Highway Research Program (NCHRP).

Recommendations. Experimental testing as described in Section 4.4.8 should be performed to determine the high-cycle elastic fatigue resistance of members reinforced with high-strength reinforcement.

3.2 Serviceability

Serviceability performance is an essential complement to the ultimate load-carrying capacity of a structure. Serviceability limit states include deflections, lateral drifts, crack control, and vibrations. These limit states are intended to avoid disruption of the intended use of the structure.

3.2.1 Deflections

ACI 318 provisions for deflections specific to flexural members are intended to provide sufficient stiffness to limit deflections that would adversely affect serviceability and strength of a structure. Deflection limits are provided as a fraction of the member span, based on use (i.e., floor or roof) and brittleness of supported materials. Member span-to-depth limits are provided with the intent of satisfying deflection limits. Methods of estimating member stiffness, and hence deflections, are also provided for cases where span-to-depth requirements are not met. Estimates for additional long-term deflections are also provided in ACI 318-14.

Some users of ACI 318 have questioned how deflections can be controlled by member depth as a proportion of span length alone, without consideration of the magnitude of the load. One explanation is to consider the issue in terms of allowable stress design. One-way slab and beam depth limits were first introduced in the 1963 edition of ACI 318, and were intended for allowable stress design using Grade 40 reinforcement. Deflections are directly related to curvature. Assuming plane sections remain plane and concrete cracks, curvature equals the change in strain divided by the depth. The change in strain can be shown to vary between approximately 0.001 and 0.0015, regardless of the magnitude of the load. The change in strain varies over a small range because: (1) the concrete stress was limited to $0.45 f_c'$; (2) the potential variation in typical concrete strengths was narrow at that time (i.e., about 3,000 psi to 5,000 psi); and (3) the reinforcement stress, and thus strain, was essentially constant for all members (i.e., the stress limit of 20 ksi was used to design most members). Members with greater design loads required proportionally greater depth.

The concept of using depth as a proportion of span length to control deflection is less easily explained in terms of strength design. The change in strain, thus curvature, is greater, and not all members have the same stress in the reinforcement at service level loads. The greater range of stress and strain makes the use of depth alone to control deflection less valid, but not excessively so. In ACI 318-63, the tables could only be used for strength design if the net reinforcing ratio was less than $0.18 f_c' / f_y$ and the reinforcement was Grade 40 or less.

In the 1971 edition of ACI 318, the table for minimum beam and one-way slab depths increased the required depths by about 25%, and a footnote was added that required adjustments for members with specified yield strengths that were not equal to 60 ksi. The adjustment for Grade 80 reinforcement required 20% greater depth, and the adjustment for Grade 40 reinforcement reduced the depth to be essentially the same as in ACI 318-63. The required depths, and the footnote, have remained essentially the same in all subsequent editions of ACI 318, including ACI 318-14 (in Table 7.3.1.1 for one-way slabs and Table 9.3.1.1 for beams) even though reductions in load factors have increased service level reinforcement stresses, leading to greater curvatures.

In lieu of using minimum depth limits for controlling deflections, deflections can be computed and compared to allowable deflections. Equation 24.2.3.5a in ACI 318-14, reproduced here as Equation 3-1, is permitted to be used as the effective moment of inertia for computing immediate deflections. This equation was developed by Branson (1977) and Branson and Trost (1982), and appears in ACI 435R-95, *Control of Deflection in Concrete Structures* (ACI, 1995):

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \qquad (3\text{-}1)$$

where I_e is the effective moment of inertia, I_g is the moment of inertia of the gross concrete section, I_{cr} is the moment of inertia of the cracked transformed concrete section, M_{cr} is the cracking moment, and M_a is maximum moment due to service loads. Equation 3-1 yields a single effective moment of inertia that was calibrated to represent the entire length of a simply supported flexural member containing both uncracked and cracked regions, and allows for deflection calculation using elastic beam equations. For continuous spans, the effective moment of inertia used for the entire span is typically taken as the average of the moment of inertia for the negative moment region(s) and the positive moment region.

The current methodology for controlling or computing deflections presents several questions when using high-strength reinforcement:

- Are the minimum span-to-depth tables of ACI 318 still appropriate?
- Is the equation for effective moment of inertia valid for high-strength reinforcement?
- Is the time-dependent deflection factor, λ_Δ (Section 24.2.4.1.1 in ACI 318-14), which is not reflective of reinforcement yield strength, valid for high-strength reinforcement?

With the use of high-strength reinforcement, it is expected that the amount of required flexural reinforcement will decrease, resulting in lower reinforcement ratios, ρ. The current design tables provide a multiplier, $0.4 + f_y / 100{,}000$ to account for reinforcement grades other than Grade 60; however, this expression is currently limited to Grade 80 reinforcement. It is expected that flexural members designed using high-strength reinforcement will be subjected to higher reinforcement stresses and greater curvatures. Therefore, an extension of the current approach may not be valid and requires further investigation.

Although Equation 3-1 accounts for the reinforcement ratio through I_{cr}, the overall expression also considers tension stiffening, which is the blending of I_{cr} and I_g to provide an effective stiffness somewhere between these two values. For low reinforcement ratios, it has been shown that Branson's equation overestimates tension stiffening (Bischoff, 2007). Therefore, an alternative expression has been developed (Bischoff, 2007; Bischoff and Scanlon, 2007) to more accurately account for this effect. The equation developed by Bischoff, reproduced here as Equation 3-2, has been shown to provide much more accurate results, even with very low effective reinforcement ratios that are produced by beams reinforced with fiber reinforced polymer (FRP) bars:

$$I_e = \frac{I_{cr}}{\left[1 - \eta\left(\frac{M_{cr}}{M_a}\right)^2\right]} \quad (3\text{-}2)$$

where:

$$\eta = 1 - \frac{I_{cr}}{I_g}$$

For this reason, ACI Committee 440 has adopted this equation for the estimation of deflection of FRP-reinforced members, and it will appear in the next version of the ACI 440.1R report. Specifically for high strength reinforcement, ACI ITG-6R-10, also presents Equation 3-2 as an appropriate approach, and this expression will also be included in the next version of the ACI 435R report on control of deflections. Because of issues related to the use of low reinforcement ratios with high-strength reinforcement, it is expected that a change from the current ACI 318 effective stiffness equation will be required, and it appears that the Bischoff expression will be a suitable replacement.

Regardless of the procedure used to estimate deflections (Equation 3-1 or Equation 3-2), deflection calculations depend on the computation of the cracking moment, M_{cr}. Although the cracking moment is typically computed using a modulus of rupture equal to $7.5\sqrt{f_c'}$, studies have indicated that improved deflection calculations are possible using a reduced value (i.e., as low as half the typical value). Lower values have been considered to be necessary to account for shrinkage, as well as restraint, which can reduce the cracking moment of a flexural member. In some research, the use of a lower modulus of rupture has been suggested to offset the over-estimation of tension stiffening that results from the use of Equation 3-1.

The appropriateness of the long-term deflection factor applied to the immediate deflection of members reinforced with high-strength reinforcement should also be considered. Because this factor is primarily influenced by creep in the concrete, it is not likely that changes will be required. However, with the use of potentially lower reinforcement ratios made possible through the use of high-strength reinforcement, higher curvatures will be possible, which may potentially increase stresses in the compression zone, leading to higher overall creep deflections. It seems appropriate that long-term deflections be evaluated to ensure appropriateness of the current time-dependent factor approach.

Although ACI 318 provides requirements for the calculation and control of flexural deflections, it is silent on the axial shortening of compression members. Often taken simply as PL/AE, (where P is the axial load, L is the length of the member, A is the area of the member, and E is the elastic modulus) the interaction of concrete and reinforcing bars strained equally, but with different yield strains, creates a more complex behavior. Furthermore, the time-dependent creep strain of column concrete is not addressed in ACI 318. The time-dependent creep factor for flexural members (Section 24.2.4.1.1 in ACI 318-14) is often used for compression members for lack of other guidance. Improved understanding regarding the shortening of compression members seems warranted, especially with the use of high-strength reinforcement. With high-strength reinforcement, column cross-sections may be reduced and axial shortening, especially long-term shortening, may increase.

Recommendations. An analytical study as described in Section 4.5.1 should be performed to investigate deflections in flexural members reinforced with high-strength reinforcement. Analytical results should be compared to tests of beams described in Section 4.4.2.

3.2.2 Drift

Acceleration and displacement response of structures due to service level lateral loads can affect the comfort of building occupants and damage to finishes, if excessive. In general, this is only a concern in taller reinforced concrete buildings subjected to wind loads. Drift is computed as part of the structural analysis, and is influenced primarily through the selection of the flexural stiffness of the members. With the use of high-strength reinforcement, a reduction in the flexural stiffness of members is expected, which can influence drift.

Recommendations. Additional detailed discussion on the influence of high-strength reinforcement on flexural stiffness is provided in Section 3.5.1. An analytical study should be performed as described in Section 4.8.1 to develop guidance on the selection of flexural stiffness, EI, for use in analyzing structures at service level loads.

3.2.3 Crack Control

Since 1971, ACI 318 has required the control of crack widths in flexural members. Explicit calculation to evaluate crack widths was added as a result of the introduction of Grade 60 reinforcement. At the time, the z-factor method was incorporated, which was based on a crack width calculation equation developed by Gergely and Lutz (1968). This equation was derived from test results focused on service level stresses that could develop using Grade 60 reinforcement, and the maximum stress considered was 50 ksi. In the 1999 edition of ACI 318, the approach used to control cracking was changed, and was based on a physical model developed by Frosch (1999). The physical model considers bar strains; therefore, it is expected that this approach is applicable for high-strength reinforcement (Frosch, 2001). It should be noted, however, that the maximum stress considered in the evaluation of the model was 50 ksi.

Cracking along the side faces of beams also requires control. The approach used in ACI 318 is consistent with that used along the tension face, and is based upon the same physical model (Frosch, 2002). Therefore, it is expected that this approach is also applicable for high-strength reinforcement. It should be noted, however, that the maximum stress considered in evaluating the accuracy of the model was 40 ksi. Therefore, evaluation of the expressions with higher grade reinforcement is warranted.

The control of cracking is expected to become even more important with the use of high-strength reinforcement. Therefore, it is important that the physical model be evaluated considering test results with higher stress ranges. In addition, it will likely be desirable to reformat the ACI 318 design expression so that the default reinforcement service stress (40 ksi) for which the equation was developed changes, or the expression should be reformatted so that the equation is not presented based on a default stress. Overall, it is expected that increased service stresses will require reduced bar spacing, which can be achieved with the same amount of reinforcement using smaller bar sizes.

Crack control is also required in ACI 318 for members designed using strut-and-tie modeling. Specifically, a minimum of 0.3% distributed reinforcement crossing the strut axis is required. According to ACI Committee 318, "This reinforcement will help control cracking in a bottle-shaped strut and result in a larger strut capacity than if this distributed reinforcement was not included."

As previously mentioned in Section 3.1.2.4, Birrcher et al. (2013) conducted twelve full-scale tests to evaluate the minimum web reinforcement requirements for deep beams. Although this study was focused on deep beams, the results can be considered appropriate for considering minimum distributed reinforcement for strut-and-tie modeling. Results indicated that a larger amount of web reinforcement was needed to control diagonal crack widths than was needed to provide shear capacity. Based on strength and serviceability considerations, this study concluded that a minimum reinforcement ratio of 0.3% in each orthogonal direction is appropriate, which is consistent with the amount of reinforcement currently specified for distributed reinforcement when strut-and-tie models are used. Although Grade 60 reinforcement was used in the testing program, it is expected that higher strength reinforcement would not impact this minimum requirement.

Recommendations. Further research is recommended to evaluate cracking behavior in flexural members when high-strength reinforcement is utilized, and a proposed research program is outlined in Section 4.5.2.

Considering crack control for structures designed using strut-and-tie models, the current minimum reinforcement amounts are considered appropriate for use with high-strength reinforcement, and no further research is recommended at this time. Furthermore, a code change to this minimum requirement is not considered necessary.

3.3 Reinforcement Limits

Reinforcement limits for concrete members are based on objectives of strength, serviceability, and constructability. The influence of these three objectives is handled separately for each primary member type.

3.3.1 Beams

Current ACI 318 provisions for minimum reinforcement in flexural members are intended to provide flexural strength of the cracked section that exceeds the flexural strength of the uncracked section (i.e., cracking moment). This is a critical limit state that protects against sudden collapse of flexural members in the event of loading beyond the cracking moment.

The current equation for minimum area of flexural reinforcement in Section 9.6.1.2 of ACI 318-14, reproduced here as Equation 3-3, already addresses reinforcement with variable yield strength:

$$A_{s,\min} = \frac{3\sqrt{f_c'}b_w d}{f_y} \tag{3-3}$$

where f_c' is specified concrete compressive stress, f_y is the specified yield strength of the reinforcement, b_w is the width, and d is the depth to the centroid of longitudinal tension reinforcement. However, this equation is only applicable to rectangular beams, and was intended to be conservative. It is possible that this equation could be eliminated, and a minimum value could be directly specified for the ratio $\phi M_n/M_{cr}$, where ϕM_n is the design flexural strength and M_{cr} is the theoretical cracking moment. This would resemble current requirements for prestressed members, in which the cracked section flexural capacity is 120% of the uncracked flexural capacity (in Sections 7.6.2.1, 8.6.2.2, and 9.6.2.1 of ACI 318-14). These generalized requirements inherently include, and consider, high strength reinforcement in the limit state. Strength-based testing would not be required to make such a change.

Recommendations. Provisions for minimum reinforcement in flexural members are currently being reworked by ACI Committee 318. If a direct solution (such as the current approach in Section 9.6.1.2 of ACI 318-14) is used, an analytical study as described in Section 4.6.1 should be performed. If an approach similar to current requirements for prestressed members (based on a multiple of the uncracked flexural capacity) is used, no further study is recommended or required at this time.

3.3.2 Slabs and Footings

ACI 318-14 provisions exempt slabs and footings of uniform thickness from satisfying strength-based minimum flexural reinforcement requirements intended to ensure that $M_{n,cracked} > M_{n,uncracked}$. This exemption is based on the expected capacity for redistribution of forces in these elements. Instead, minimum reinforcement for slabs and footings is based on shrinkage and temperature requirements, but with the requirement that this reinforcement be provided entirely on the tension face of the member.

The shrinkage and temperature provisions for slabs in Section 8.6.1.1 of ACI 318-14 already addresses reinforcement with variable yield strength. Reproduced here as Equation 3-4, the minimum ratio of reinforcement area to gross concrete area is:

$$\frac{0.0018x60}{f_y} \tag{3-4}$$

where f_y is the specified yield strength of the reinforcement in ksi. Yield strength in this formula is based on the extension under load method for a strain of 0.35%. This measurement of yield strength recognizes that reinforcement with a nonlinear stress-strain curve shape prior to yield does not provide as much resistance to crack control. ACI 318 eliminated this approach to measurement of yield strength, and uses the 0.2% offset method instead. Current provisions for minimum slab reinforcement will need to be revisited, and possibly revised, considering this change in the definition of yield strength, which eliminates a means of differentiating reinforcement with and without nonlinear stress-strain curve behavior prior to the defined yield strength.

Recommendations. If a strength-based approach for determining minimum flexural reinforcement in slabs and footings is implemented, an analytical study as described in Section 4.6.2 should be performed.

3.3.3 Columns

ACI 318 requires a minimum amount of reinforcement in all columns, specified as 1% of the gross section area, and this minimum has been constant since the original 1936 edition of the code, ACI 501-36T, *Building Regulations for Reinforced Concrete* (ACI, 1936). The commentary to ACI 318-11 states that "Reinforcement is necessary to provide resistance to bending, which may exist regardless of analytical results, and to reduce the effects of creep and shrinkage of the concrete under sustained compressive stresses. Creep and shrinkage tend to transfer load from the concrete to the reinforcement, and the resultant increase in reinforcement stress becomes greater as the reinforcement ratio decreases. Therefore, a minimum limit is placed on the reinforcement ratio to prevent reinforcement from yielding under sustained service loads (Richart, 1933)." It should be noted that it is possible for a column to have less than 1% reinforcement. When a column is oversized due to architectural considerations, the code permits consideration of a reduced area section, but not less than one-half the total area. As a result, reinforcement ratios as low as 0.5% are possible.

A research project conducted by Ziehl et al. (1998; 2004) assessed the feasibility of reducing current ACI 318 and AASHTO LRFD Bridge Design Specification requirements for minimum longitudinal reinforcing steel in columns. Considering that current requirements are based primarily on research conducted in the 1920s and 1930s, on concrete with compressive strength generally less than 5,000 psi, research was conducted to determine the effects of present-day construction materials on minimum reinforcement requirements. The research program included: (1) fabrication and long-term loading and monitoring of 24, 8-inch diameter by 4-foot long reinforced concrete column specimens; (2) fabrication of reduced-humidity enclosures for storage of all specimens throughout the test program; and (3) long-term monitoring of 14 unloaded companion specimens. Test variables included concrete strength and reinforcement ratio, and Grade 60 reinforcement was used in all specimens. All loaded specimens were subjected to a nominal compressive force of $0.40 f'_c A_g$. It was found that for reinforcement ratios ranging from 0% to 0.72%, all of the specimens with high-strength concrete, and most with normal strength concrete, experienced time-dependent axial strains that exceeded the yield strain of the reinforcement (0.00207). Therefore, it was concluded that a reduction of the current ACI 318 requirement for minimum longitudinal reinforcement was not justified because “passive yielding, based on the nominal yield strength of the longitudinal reinforcement, occurred in all high-strength specimens, and would likely have occurred in most of the other specimens had loading continued for another year.”

Considering that the yield strain of high-strength reinforcement will be higher, there is a potential that the minimum reinforcement ratio can be reduced. However, the economic benefit of reductions below 1% may not be significant. Furthermore, decreased reinforcement ratios likely will increase creep deformation, and have the potential to increase column shortening. Overall, it seems appropriate to maintain current requirements for minimum reinforcement ratio in columns.

Recommendations. No further study is recommended or required at this time, and a code change is not considered necessary.

3.3.4 Walls

ACI 318 requirements for walls reflect minimum reinforcement ratios that depend on yield strength, f_y, as a step function. One limit exists for reinforcement with f_y less than 60 ksi, and a smaller minimum ratio is allowed for reinforcement with f_y “not less than” 60 ksi. Although this trend could be extrapolated to higher strength reinforcement, and correspondingly lower reinforcement ratios, the current limits are deemed to be sufficiently low.

As discussed in Wood (1989), very low reinforcement ratios, particularly at the base of buildings, can lead to a localization of strain in longitudinal reinforcement when walls crack. Brittle failures may occur at low drift ratios, particularly when the cracked flexural capacity of the wall is less than its uncracked flexural capacity (Davey and Blaikie, 2005). Such failures are possible when a single flexural crack occurs, and high-strain low-cycle fatigue initiates longitudinal bar fracture.

Recommendations. No further study is recommended or required at this time, and a code change is not considered necessary.

3.4 Detailing and Other Design Considerations

Detailing requirements include bar bending, bar spacing, development and splice lengths of straight bars, development lengths of hooked and headed bars, continuity and termination of bars, and concrete cover for reinforcement. Structural detailing and member connectivity need to be examined when using high-strength reinforcing bars as concrete reinforcement.

3.4.1 Bend Test Requirements and Minimum Bend Diameters

Reinforcing bars produced in accordance with ASTM standard specifications must conform to specific bend test requirements. Bend test requirements define the degree of bending and the pin diameters about which reinforcing bars must be bent without experiencing cracking or fracture.

The minimum percentages of elongation at fracture in an 8-inch gauge length required by ASTM A615, ASTM A706, and ASTM A1035 are summarized in Table 3-1. ACI 318-14 minimum bend diameters for hooks, and the pin diameters for bend tests required by ASTM specifications, are summarized in Table 3-2. Current use of ASTM A1035 Grade 100 and Grade 120 bars as transverse reinforcement has adopted the minimum bend diameters commonly used for Grade 60 reinforcement.

Table 3-1 Minimum Percentage of Fracture Elongation for ASTM A615, ASTM A706, and ASTM A1035 Reinforcing Bars(1)

Bar Size (No.)	A615 Grade 40	A615 Grade 60	A615 Grade 75	A706 Grade 60	A706 Grade 80	A1035 Grade 100	A1035 Grade 120
3	11	9	–	14	12	7	7
4 and 5	12	9	–	14	12	7	7
6	–	9	7	14	12	7	7
7 and 8	–	8	7	12	12	7	7
9, 10, and 11	–	7	6	12	12	7	7
14 and 18	–	7	6	10	10	6	–

Notes: (1) All values correspond to percentage of elongation in an 8-inch gauge length.

Table 3-2 Minimum Bend Diameter of Hooks in ACI 318 and Bend Test Requirements for ASTM A615, ASTM A706, and ASTM A1035 Reinforcing Bars(1)

	Minimum Bend Diameter of Hooks per ACI 318-14(2)		Pin Diameter for Bend Test per ASTM Specifications						
Bar Size (No.)	Stirrups, Ties, and Hoops	All Other Bars	A615 Grade 40	A615 Grade 60	A615 Grade 75	A706 Grade 60	A706 Grade 80	A1035 Grade 100	A1035 Grade 120
3, 4, and 5	$4d_b$	$6d_b$	$3\frac{1}{2}d_b$	$3\frac{1}{2}d_b$	–	$3d_b$	$3\frac{1}{2}d_b$	$3\frac{1}{2}d_b$	$3\frac{1}{2}d_b$
6	$6d_b$	$6d_b$	$5d_b$	$5d_b$	$5d_b$	$4d_b$	$5d_b$	$5d_b$	$5d_b$
7 and 8	$6d_b$	$6d_b$	–	$5d_b$	$5d_b$	$4d_b$	$5d_b$	$5d_b$	$5d_b$
9, 10, and 11	–	$8d_b$	–	$7d_b$	$7d_b$	$6d_b$	$7d_b$	$7d_b$	$7d_b$
14 and 18	–	$10d_b$	–	$9d_b$	$9d_b$	$8d_b$	$9d_b$	$9d_b$	$9d_b$

Notes: (1) d_b: nominal bar diameter.
(2) ACI 318-14 allows up to Grade 100 for stirrups, ties, and hoops; all other bars are limited to Grade 80.

Kudder and Gustafson (1983) reported the results of 945 bend tests and 540 tension tests performed on five sizes of Grade 60 reinforcing bars. Bend test data showed that for bars with a percentage of elongation at least equal to ASTM specified values, the probability of a bend test failure was very small (0.02 percent or less). They concluded that "there is a very low likelihood of a failure occurring at a bend diameter equal to, or larger than the minimum bend diameter required by the ACI Building Code."

Stecich et al. (1984) reported the results of 254 bending and straightening tests conducted on Grade 60 reinforcing bars. The major variables were bend diameter, bar size, supplier, bend axis, and temperature. Three bar sizes (No. 5, No. 8, and No. 11) were included in the tests. Smaller bars performed better, and breakage and cracking were more likely to occur when bars were straightened at cold temperatures. They concluded that field bending and straightening of reinforcing bars up to No. 11 should be permitted, as long as the bars are bent about the weak axis up to 90 degrees using the minimum bending bar diameters of ACI 318, and are straightened at normal temperatures.

Recommendations. Experimental testing and statistical analysis of data as described in Section 4.3.2 should be performed to determine the mechanical properties of high-strength reinforcing bars, including bend test requirements and minimum bend diameters.

3.4.2 Transverse Reinforcement Spacing

Transverse reinforcement serves different purposes for different members and systems. In this *Roadmap*, transverse reinforcement spacing requirements are addressed separately for: (1) members that are designed to resist gravity and wind loads, or members resisting seismic loads in ordinary seismic-force-resisting systems; and (2) members resisting seismic loads in intermediate and special seismic-force-resisting systems, or members in structures assigned to Seismic Design Category (SDC) D, E, or F, which are not specifically designated as part of the seismic-force-resisting system.

3.4.2.1 Spacing of Transverse Reinforcement in Members Resisting Gravity and Wind Loads, or Seismic Loads in Ordinary Seismic-Force-Resisting Systems

Spacing of transverse reinforcement can be controlled by shear strength requirements, shear reinforcement spacing requirements, requirements for spirals to provide confinement to the column core, or detailing requirements for spirals or ties. This section focuses on the detailing requirements for spirals and ties in members resisting gravity and wind loads, and members in ordinary seismic-force-resisting systems. Required spacing for shear resistance and confinement are not considered in this section.

The design of spirally reinforced columns is based on the assumption that when the cover spalls, the concrete core will remain confined so that it can maintain strength at least equal to the strength of the column prior to spalling. To meet this intent, spirals must also prevent longitudinal bars from buckling. In Section 25.7.3.1 of ACI 318-14, maximum clear spacing for spiral reinforcement is 3 inches. For comparison, the recommendation for hoop spacing of Grade 100 and Grade 120 longitudinal bars in NIST GCR 14-917-30 is $4d_b$, where d_b is the diameter of the longitudinal bar. This recommendation would result in 3-inch center-to-center spacing for No. 6 bars. For gravity and wind loading, the 3-inch maximum clear spacing requirement for spirals is likely adequate for longitudinal bars up to Grade 120.

The center-to-center spacing required for column ties placed in accordance with Section 25.7.2.1 of ACI 318-14 is no greater than the minimum of: (1) $16d_b$ where d_b is the smallest longitudinal bar in the column; (2) $48d_b$ where d_b is the diameter of the tie bar; and (3) the smallest dimension of the column section. These tie spacing requirements have been unchanged since the original 1936 version of the code, ACI 501-36T. An explanation for tie requirements that are close to the requirements in ACI 318-14 appears in *Standard Building Regulations for the Use of Reinforced Concrete* (NACU, 1910), which states that longitudinal bars need to be sufficiently held in place until the concrete is set.

The commentary to ACI 318-14 indicates that Pfister (1964) performed tests that showed that, at the tie spacing used, there was no appreciable strength difference between columns with and without ties. Following this research, tie spacing did not change in the 1971 edition of ACI 318, but the required diameter of the ties was increased and related to the longitudinal bar size, and the requirement that each longitudinal bar be supported by transverse reinforcement was relaxed. Bars within 6 inches of laterally supported bars no longer needed to be supported by transverse reinforcement. The current requirements for ties in ACI 318-14 are the same as those in ACI 318-71.

The maximum allowable spacing for transverse reinforcement in columns with high-strength longitudinal reinforcement may not require change, although if smaller diameter high-strength reinforcing bars are used, the likelihood of bar buckling could be increased, and there could be some need to reduce the spacing. Consideration should also be given to imposing new spacing requirements for transverse reinforcement in beams and columns of ordinary moment frames, and at the ends of ordinary structural walls.

It should be noted that the tests serving as a basis for this information were performed on concentrically loaded columns, and did not account for sustained loading and creep effects that can be detrimental to bar stability. Subjecting columns to moderate lateral deformations (i.e., wind or low seismicity) after creep effects may cause bars to buckle under current spacing requirements for ties.

Recommendations. No further experimental investigation or engineering study is recommended at this time. Requirements for revised spacing of transverse reinforcement in members resisting gravity and wind loads could be decided based on current information and consensus opinion. This should also include consideration of revised spacing of transverse reinforcement in members of ordinary seismic-force-resisting systems.

3.4.2.2 Spacing of Transverse Reinforcement in Members of Intermediate and Special Seismic-Force-Resisting Systems

Members of intermediate and special seismic-force-resisting systems, as well as members in structures assigned to SDC D, E, or F that are not specifically designated as part of the seismic-force-resisting system, require transverse reinforcement to maintain an appropriate level of ductility when subjected to repeated loading beyond yield. Transverse reinforcement in the form of hoops or spirals increases ductility by confining the concrete and restraining longitudinal bars from buckling.

The use of high-strength reinforcement to confine concrete has been studied in the past (Budek et al., 2002; Muguruma et al., 1990; and Sugano et al., 1990), and ACI 318-14 allows the use of Grade 100 reinforcement to provide confinement in columns and wall boundary elements in special seismic systems. Grade 100 is also allowed for spirals providing confinement in columns. Existing research could be re-evaluated to determine if Grade 120 reinforcement could be similarly used. If necessary, additional tests could be performed to confirm the effectiveness of Grade 120 reinforcement for confinement of concrete. Transverse reinforcement used to confine the concrete core generally requires spacing that is less than the maximum allowed spacing, so tests to investigate confinement would be different from tests used to determine the maximum allowable spacing of transverse reinforcement.

Transverse reinforcement in columns and boundary elements of special seismic systems is also intended to control bar buckling. ACI 318-14 specifies the maximum spacing of transverse reinforcement in the zone of expected column yielding to be the smallest of: (1) one-quarter the minimum member dimension; (2) $6d_b$, where d_b is the diameter of the smallest longitudinal bar; and (3) s_o, which depends on the spacing of the vertical bars, but is not more than 6 inches. For wall boundary elements, item (1) is changed to one-third the thickness of the boundary element. The spacing, s_o, in item (3) is required for confinement, but the maximum limit of 6 inches provides additional restraint for buckling of bars larger than No. 8.

The maximum spacing of transverse reinforcement is expected to require adjustment for restraining high-strength longitudinal reinforcement. Both the spacing and the stiffness of the transverse reinforcement are important (Tanaka, 1990; Restrepo-Posada, 1992; Rodriguez et al., 1999; Wang and Restrepo, 2001; Moyer and Kowalsky, 2003). Requirements for maximum spacing of transverse reinforcement for buckling restraint of high-strength longitudinal bars was studied in NIST GCR 14-917-30. An analytical approach was used to determine the maximum spacing required to obtain bar-buckling restraint equivalent to that provided by transverse reinforcement spaced at $6d_b$ restraining Grade 60 longitudinal reinforcement. Results indicated that the spacing of transverse reinforcement should be reduced to $5d_b$ for Grade 80 longitudinal reinforcement and $4d_b$ for Grade 100 and Grade 120 longitudinal reinforcement.

Testing is needed to confirm these results, or to determine alternative maximum spacing requirements. Testing is also needed to determine if every longitudinal bar should have lateral support provided by a crosstie. Currently, crossties are only required on every other longitudinal bar (for bars less than 6 inches apart). Past tests have shown that lateral support on every bar provides better confinement, and may be necessary to provide buckling restraint.

Several limits are imposed for the maximum spacing of transverse reinforcement in beams and columns of intermediate moment frames. These limits are not based on test data. Requirements are less than requirements for special moment frames because the expected inelastic demands are lower. However, maximum spacing limits of $8d_b$ or 12 inches are likely to require change with the use of high-strength longitudinal reinforcement. Testing will be needed to justify limits for maximum spacing of transverse reinforcement if Grade 100 and Grade 120 longitudinal bars are used in intermediate moment frames.

Consideration should also be given to imposing new or revised maximum spacing requirements for transverse reinforcement at the end regions of special structural walls where special boundary elements are not required.

Recommendations. Experimental testing as described in Sections 4.9.3, 4.9.4.1, 4.9.4.2, 4.9.7, and 4.9.10 should be performed to evaluate the required spacing of transverse reinforcement in beams, columns, and walls in seismic-force-resisting systems.

3.4.3 Head Size and Attachment for Headed Deformed Bars

Questions that arise in the fabrication of headed deformed bars with high-strength reinforcement include: (1) restrictions on the type of head; (2) the required size of the head; and (3) the required strength of the attachment to the head. Answers to these questions are expected to be especially important for special moment frames and shear walls subjected to seismic loading, and high-strength reinforcement with rounded stress-strain curves.

A three-year, industry-supported research study is now in progress at the University of Kansas. Ongoing tests indicate that headed bars fabricated with high-strength reinforcement can provide satisfactory performance at yield strengths up to 120 ksi. In the case of monotonic loading, tests results indicate that the behavior of headed bars with high-strength reinforcement is not markedly different from the behavior of headed bars with conventional reinforcement. To date, test specimens under monotonic loading have not indicated a need for larger heads, and no problems have been observed based on the type of head used, or the presence of a rounded stress-strain curve. Producers have indicated, however, that the material used for the head may need to be changed to achieve tensile strengths above 125 ksi.

Recommendations. Pending confirmation and acceptance of test results from the University of Kansas, no additional experimental investigation or engineering study is recommended at this time for head size or attachment of headed deformed bars under monotonic loading conditions.

For cyclic loading conditions, the potential need for different head size or attachment of headed deformed bars is discussed in connection with beam-column joints in Section 3.6.3.5, and recommended testing is included as part of exterior joint studies in Section 4.9.4.4.

3.4.4 Development and Splice Lengths

Reinforcing bars are deformed, which limits slip and provides attachment to the concrete. Attachment is also provided by hooks and other devices, such as heads, if the available straight length is insufficient for development of the bar. Design procedures used for conventional reinforcement do not necessarily reflect the development and splice length needs associated with high-strength reinforcement.

3.4.4.1 Straight Bar Development Length

Test results on the development and splice strengths of reinforcing bars with conventional deformation patterns are available for No. 3 through No. 11 bars. The database also includes two No. 14 bars not confined by transverse reinforcement. Figure 3-5 and Figure 3-6 show the combinations of developed bar stress and concrete compressive strength for tests without confining transverse reinforcement (Figure 3-5) and with confining transverse reinforcement (Figure 3-6).

Although most tests have been for developed bar stresses below 80 ksi, there are a significant number of tests available for bar stresses in excess of 80 ksi; enough to establish the adequacy of design expressions for concrete compressive strengths below 10,000 psi. There are insufficient data, however, for bars above 80 ksi in higher strength concrete (i.e., above 10,000 psi). Specimens used in splice tests of high-strength reinforcement have had covers of at least one bar diameter, and spacing between splices of at least two bar diameters. To complete the results, test specimens with bar spacing down to the greater of one bar diameter or 1 inch are needed.

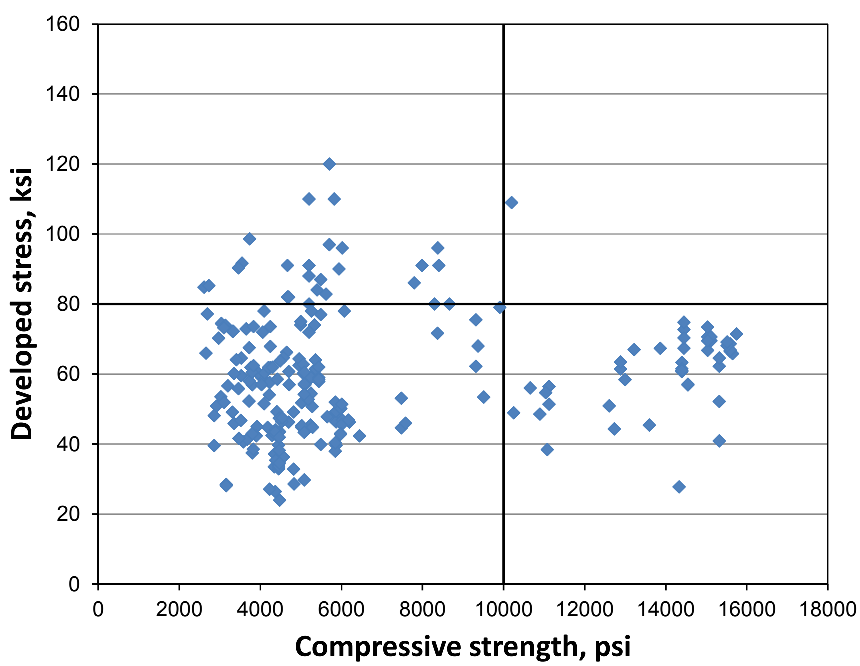


Figure 3-5 Combinations of developed bar stresses and concrete compressive strengths for development and splice tests on bars without confining transverse reinforcement.

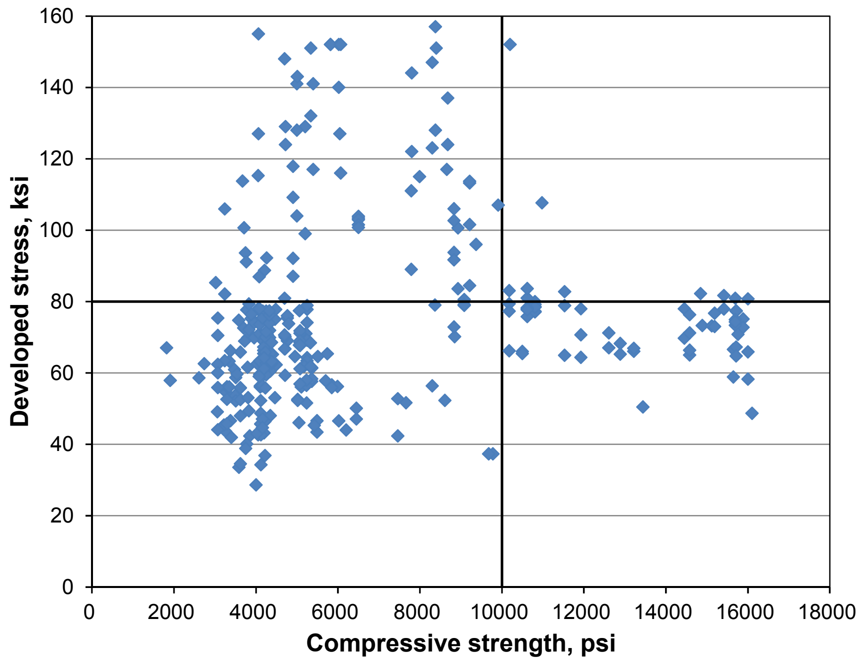


Figure 3-6 Combinations of developed bar stresses and concrete compressive strengths for development and splice tests on bars with confining transverse reinforcement.

Development and splice length provisions in ACI 318 were based on tests in which bar stresses were, for the most part, less than 60 ksi (Orangun et al., 1975). ACI 318 equations for development length are simplified in that development (or splice) length is proportional to bar stress. That simplification is conservative for bars with yield strengths below 60 ksi. The relationship becomes progressively less conservative, and even unconservative, as the yield strength increases above 60 ksi. This is especially true for bars not confined by transverse reinforcement. There is, however, a direct relationship between the amount of confining transverse reinforcement and the increase in stress developed in a spliced or developed bar due to the addition of the confining reinforcement (Zuo and Darwin 2000; ACI, 2003). This generally results in adequately conservative results when the ACI provisions are applied to bars confined by transverse reinforcement, and suggests that transverse reinforcement should be required for high-strength reinforcement if current ACI equations are retained.

It was originally proposed that the spacing between bars and minimum cover on bars be larger so that development lengths could be shorter. To keep sections as small as possible, bar spacing and cover limits were not increased. However, with high-strength reinforcing bars, current spacing and cover requirements could result in the need for a correspondingly larger amount of transverse reinforcement.

It has been demonstrated (Seliem et al., 2009; ACI, 2010a; Choi et al., 2014) that the design expressions developed by ACI Committee 408 (ACI, 2003) are appropriately accurate and conservative for development and splicing of bars with stresses as high as 150 ksi.

No development and splice length tests have been reported on epoxy-coated bars at stresses above 80 ksi. Tests in this area are needed.

Recommendations. Additional development and splice length tests on straight high-strength reinforcing bars should be performed as described in Section 4.7.1 in concrete with compressive strengths greater than 10,000 psi, spaced no more than the greater of 1 inch or one bar diameter apart, and coated with epoxy.

3.4.4.2 Hooked Bar Development Length

Part of a research study now in progress at the University of Kansas includes testing to determine the development length of high-strength hooked reinforcing bars. Prior to this testing, available data on the development of hooked bars were limited, and involved simulated beam-column connections in which one or two reinforcing bars with standard (or nonstandard) hooks were pulled out of a concrete block. In these tests, compression was applied to the surface of the column specimen to simulate the compression region of a beam framing into the column.

Prior to the University of Kansas tests, only one series of tests had been performed on high-strength hooked reinforcing bars. In these tests (Shahrooz et al., 2011), individual hooked bars were cast well away from member boundaries. The application of these tests is therefore limited to hooked bars anchored in deep foundations, away from the exterior face of the concrete.

To date, tests at the University of Kansas indicate that the current ACI 318 provisions for hooked bars become progressively less conservative (and even unconservative) as the size of the bar and the concrete compressive strength increase. In addition, tests on closely spaced hooked bars indicate that the current provisions become less conservative as the bar spacing decreases.

The scope of the University of Kansas testing does not include the use of staggered hooks, and no data are available on the development of epoxy-coated, high-strength, hooked reinforcing bars.

Recommendations. Additional tests on closely spaced, staggered hooked bars should be performed as described in Section 4.7.2. Tests on epoxy-coated hooked reinforcing bars would also be useful.

3.4.4.3 Headed Bar Development Length

Until recently, only limited information has been available on the development of high-strength headed reinforcement. Test results are now just becoming available from Korea (Bang, 2014) and additional test results will soon be available as part of a research study now in progress at the University of Kansas. The Korean research, which is largely connected to the construction of nuclear power plants, has resulted in recommendations to modify the development length for headed reinforcing bars. Depending on the application, increases or decreases in ACI 318-14 development lengths have been recommended.

To date, tests at the University of Kansas indicate that some portions of the Korean recommendations may be unconservative in beam-column joints with closely spaced headed bars. A discussion of headed deformed bars in beam-column joints subjected to seismic loading is provided in Section 3.6.3.5.

Recommendations. Additional tests on closely spaced, staggered headed bars should be performed as described in Section 4.7.3.

3.4.4.4 Mechanical Splices

Before manufacturers embark on testing to validate mechanical splices for high-strength reinforcement, existing mechanical splice requirements should be revisited to determine if changes are necessary. Type 2 mechanical splices are required to develop the specified tensile strength of the bar (as opposed to the actual tensile strength). As such, Type 2 mechanical splices located in a plastic hinge region may not develop adequate strain for high-strength bars with a rounded stress-strain curve, or with an actual tensile strength that exceeds the specified tensile strength. For this reason, NIST GCR 14-917-30 introduced the concept of a Type 3 mechanical splice that would develop the actual tensile strength of the bar, and allow the bar to fracture away from the splice or be capable of developing a preset uniform strain in seismic applications calling for high ductility demands.

Generally, coupler manufacturers indicate that current designs can be adapted to high-strength reinforcement, but a new specification, or modifications to the existing specification, may be needed for mechanical splices in high-strength reinforcement, particularly if Type 3 couplers are introduced.

Recommendations. No further experimental investigation or engineering study is recommended at this time. Requirements for mechanical splices of high-strength reinforcement, and the possible need for a Type 3 mechanical splice, could be decided based on current information and consensus opinion.

If a new or revised specification is developed, coupler manufacturers will need to demonstrate the ability to comply with the required mechanical properties.

3.4.5 Bar Extensions in Slabs

Typically, longitudinal reinforcing bars extend beyond the inflection point a distance equal to the maximum of: (1) the depth to the centroid of the longitudinal reinforcement, d; (2) a multiple of the longitudinal bar diameter, $12d_b$; or (3) a portion of the span length, $l_n/16$. Higher yield strengths are likely to require longer development lengths, so it is possible that bar extensions for one-way and two-way slabs might need to be increased.

3.4.5.1 Bar Extensions in One-Way Slabs

For one-way slabs, Sections 7.7.3.8.3 and 7.7.3.8.4 in ACI 318-14 provide the requirements for minimum bar extensions beyond points of inflection for positive and negative moment reinforcement, respectively. In a continuous slab, the positive moment reinforcement must extend beyond a point of inflection the greater of d and $12d_b$. One-third of the negative moment reinforcement must extend past the point of inflection by the greater of d, $12d_b$, and $l_n/16$.

Commentary Section R12.10.3 in ACI 318-11 states the reason for bar extensions as: "The moment diagrams customarily used in design are approximate; some shifting of the location of maximum moments may occur due to changes in loading, settlement of supports, lateral loads, or other causes. A diagonal tension crack in a flexural member without stirrups may shift the location of the calculated tensile stress approximately a distance d towards a point of zero moment."

Recommendations. No further study is recommended or required at this time, and a code change is not considered necessary.

3.4.5.2 Bar Extensions in Two-Way Slabs

Section 8.7.4.1.3 and Figure 8.7.4.1.3(a) in ACI 318-14 provide requirements for minimum bar extensions in two-way slabs. These bar extensions were established considering dead and live loads acting on the slab. If the slab is part of a slab-column frame designed to resist lateral loads, these minimum bar extensions still apply, but they must also be computed based on the distribution of moments for all applicable load combinations. Bar extensions for two-way slabs first appeared in the 1956 edition of ACI 318, when Grade 60 reinforcement was not yet being specified (Erlemann, 1997 and 1999; Meinheit and Felder, 2014). Some of the requirements in the 1956 and 1963 editions of ACI 318 exceed current requirements; however, minimum bar extensions have remained unchanged since the 1971 edition of ACI 318, when Grade 60 reinforcement was being specified.

The commentaries to the 1963 and 1971 versions of ACI 318 do not indicate how the minimum bar extensions for two-way slabs were established. It is possible they were based on estimates of inflection point locations due to pattern loading, with added length to account for tensile stress shifting due to shear cracking, as described for one-way slabs in Section 3.4.5.1.

Recommendations. No further study is recommended or required at this time, and a code change is not considered necessary.

3.4.6 Horizontal Support of Offset Column Reinforcement

Longitudinal bars for square and rectangular columns, especially the corner bars, are typically offset at splices. Section 10.7.4.1 in ACI 318-14 requires that the slope of the inclined portion of an offset bar not exceed 1 in 6 relative to the longitudinal axis of the column. The force in the inclined portion of a bar has a horizontal component that must be resolved near the bends. Section 10.7.6.4.1 requires that horizontal support be provided to resolve 1.5 times the horizontal component of force in the inclined portion of the bar. This support can be provided by a portion of the floor construction or from transverse ties or spirals located within 6 inches of the bends, as required in Section 10.7.6.4.2. The 1999 edition of ACI 318 provides guidance as follows, "For practical purposes, three closely spaced ties are usually used, one of which may be part of the regularly spaced ties, plus two extra ties."

A similar requirement appears in the 1941 edition of ACI 318, except that the ties needed to be within eight longitudinal bar diameters ($8d_b$) of the bends. This requirement was changed to 6 inches in the 1971 edition of ACI 318, and for commonly used longitudinal bar sizes in columns, the 6-inch limit is more restrictive. Unfortunately, the commentary to the 1971 edition of ACI 318 provides no information regarding the change. It occurred soon after Grade 60 reinforcement became widely available, so it is possible that the change was made in recognition of the transition from structural (33 ksi) and intermediate grade (40 ksi) reinforcement to Grade 60 reinforcement.

References addressing the need to resist the horizontal component of force in inclined longitudinal bars were not identified. It is possible that requirements were based on observations of spalling of concrete cover where the horizontal component was not resolved. For equal sized bars with the same incline and bend geometry, the bar with higher yield strength will develop a larger horizontal component to be resisted, in proportion to its increase in yield strength. It will also increase the tensile stress in the concrete, which could increase spalling of concrete cover at the bends if current limits remain unchanged. With high-strength reinforcement, reducing the proximity of the ties that resist the horizontal component of force in inclined bars appears to be appropriate.

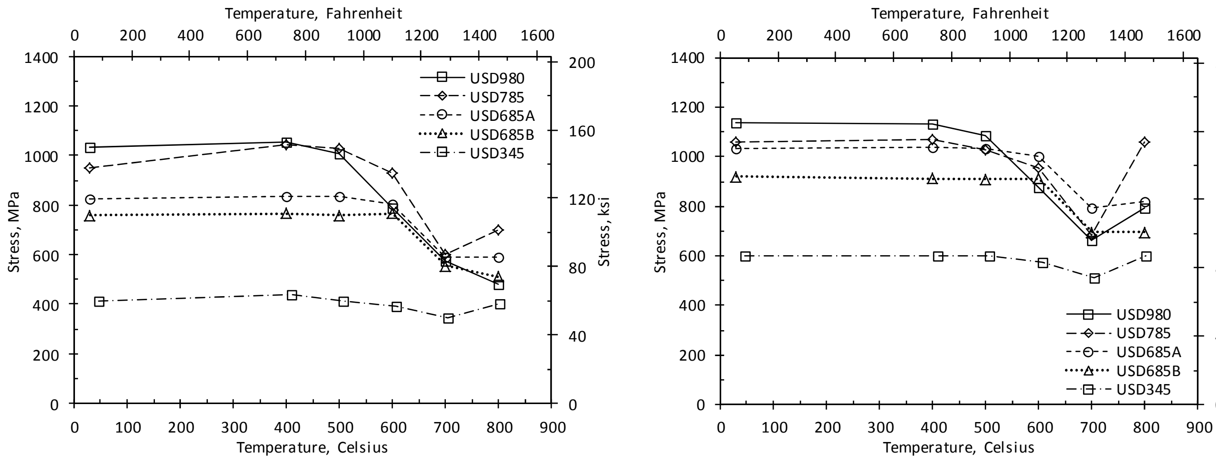
Recommendations. No experimental investigation or engineering study is recommended at this time. Requirements for reducing the proximity of the ties that resist the horizontal component of force in inclined bars could be decided based on current information and consensus opinion.

3.4.7 Cover for Fire Protection

The mechanical properties of common building materials generally decrease with elevation of temperature. A rise in temperature of concrete and steel due to fire causes a decrease in the strength and modulus of elasticity of the concrete and steel reinforcement. Some of these changes are not recoverable after subsequent cooling. The *International Building Code* (ICC, 2012) contains prescriptive requirements for building elements and contains tables describing various assemblies of building materials and finishes that meet specific fire ratings. Fire rating requirements in building codes are based on the type of occupancy and the building height and area. The fire resistance prescribed in building codes may require member dimensions (including concrete cover for reinforcement) that are significantly different from those based on ACI 318 strength design criteria.

Figure 3-7 shows the yield strength and tensile strength for various reinforcing bars after exposure to high temperature (heating and cooling). The data correspond to reinforcing bars commonly used in Japan. The figure indicates that there is a greater reduction in both the yield and tensile strengths for higher grade reinforcement. Nonetheless, at temperatures approaching 1000 degrees Fahrenheit (537 degrees Celsius) relative changes in the yield and tensile strengths are comparable to those experienced by conventional reinforcing bars. A more pronounced reduction occurs if the yield and tensile strengths are measured while the bars are being heated, as shown in Figure 3-8.

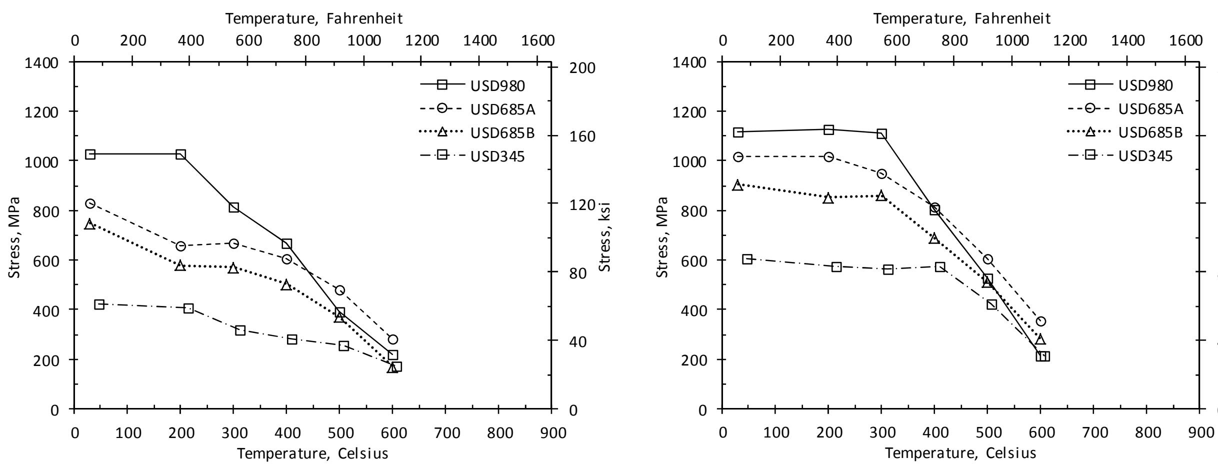
As indicated by Edwards and Gamble (1986), exposure to high temperatures tends to more severely affect the yield strength of small bars (e.g., No. 4) and the tensile strength of large bars (e.g., No. 11 bars). Because higher strength materials are impacted by a rise in temperature more than lower strength materials, the behavior of high-strength reinforcement under temperature changes, as well as cover requirements to provide adequate fire protection for high-strength reinforcement, should be investigated. Consideration should be given to the effects of bar size and the ratio of tensile-to-yield strength.



(a) Yield Strength

(b) Tensile Strength

Figure 3-7 Yield strength and tensile strength of reinforcing bars after heating and cooling (adapted from Aoyama, 2001).



(a) Yield Strength

(b) Tensile Strength

Figure 3-8 Yield strength and tensile strength of reinforcing bars at high temperature (adapted from Aoyama, 2001).

Recommendations. Experimental testing to evaluate the effects of fire on high-strength reinforcement is needed. However, fire considerations are beyond the scope of ACI 318, and is considered beyond the scope of this *Roadmap*, so testing is not recommended at this time.

3.4.8 Beam-Column and Slab-Column Joints

Beam-column and slab-column joint requirements are contained in Chapter 15 of ACI 318-14. The requirements for interior and exterior joints are treated separately.

3.4.8.1 Interior Beam-Column Joints

For interior joints of frames resisting gravity or wind loads, and frames in ordinary and intermediate systems resisting seismic forces, ACI 318-14 does not require minimum joint depths or specify the maximum bar size that can pass through the joint. Such joints are referred to as Type 1 joints in ACI 352R-02, *Recommendations for Design of Beam-Column Connections in Monolithic Reinforced Concrete Structures* (ACI, 2002b).

Bars passing through an interior joint may be in compression on one side of the joint and tension on the other side of the joint. The change in bar force through the joint depth causes bond stresses in the concrete that will increase with the use of high-strength reinforcement.

Bars passing through Type 1 joints in frames resisting gravity and wind loads are not expected to undergo inelastic deformations. For the case in which gravity loads dominate the design of the beams and columns, the change in bar stress through a joint is usually small. For frames with high-strength reinforcement in which the reinforcement is not expected to yield, deterioration within the joint due to high bond stresses is not expected, and requirements for minimum joint depth or maximum bar size passing through the joint are likely unnecessary.

However, bars passing through Type 1 joints in ordinary and intermediate moment frames resisting seismic forces are expected to undergo inelastic deformations, although the degree of inelastic activity is much less than expected in special moment frames. Higher strains due to bar yielding will result in higher bond stresses in the joints, and requirements for minimum joint depth or maximum bar size passing through the joint may be prudent. A limited number of tests is likely sufficient to investigate this issue.

Although the shear stress through interior joints can be high, ACI 318-14 does not address the shear stress in Type 1 joints directly. It does, however, require minimum transverse reinforcement, except in joints confined by beams on four sides. ACI 352R-02 provides recommendations for an explicit check of joint shear. Such a requirement, however, would be needed regardless of the yield strength of the reinforcement.

Recommendations. For Type 1 interior joints in frames resisting gravity or wind loads, no further study is recommended or required at this time, and a code change is not considered necessary.

For Type 1 interior joints in ordinary and intermediate moment frames resisting seismic forces, the possible need for requirements on minimum joint depth or maximum bar size passing through a joint should be separately considered. A small number of tests representative of intermediate moment frames should be performed as part of testing for special moment frames outlined in Section 4.9.4.3.

3.4.8.2 Exterior Beam-Column Joints

For exterior joints of frames with beam bars terminating in the joint, minimum joint depth is controlled by the depth needed to develop the hooked or headed beam bars within the joint, whether or not the bars are expected to yield. ACI 318-14 has no other specific requirements that might control the joint depth, such as a check on the joint shear. The need for such requirements should be considered with the adoption of high-strength reinforcement, and separate consideration should be given to joints with beam bars that are expected to yield, versus joints with beam bars that are not expected to yield.

In frames resisting gravity and wind loads, beam bars terminating in the joint are not expected to yield. For exterior joints in such frames, the behavior is likely to be similar regardless of the yield strength of the reinforcement. This could be investigated with a limited number of tests.

For exterior joints of ordinary and intermediate moment frames designed to resist seismic forces, the need for adjustments to joint depth, additional joint reinforcement, and limits on joint shear should be explored. In addition, the potential for multi-bar breakout failure in joints with closely spaced hooked bars should be investigated. If needed, this issue will likely be addressed by additional requirements for transverse reinforcement in the joints.

Recommendations. A small number of tests representative of intermediate moment frames should be performed as part of testing for exterior joints in special moment frames outlined in Section 4.9.4.4.

3.4.8.3 Slab-Column Joints

Slab-column joints are used in gravity framing and in slab-column ordinary and intermediate moment frames. For gravity framing, the shear strength of slab-column joints is discussed Section 3.1.2.1.2.

In ordinary and intermediate moment frames, slab-column joints should be capable of undergoing anticipated story drifts associated with seismic loading without shear failure or collapse. For intermediate moment frames, the shear strength of slab-column joints is discussed in Section 3.6.2.

Recommendations. For gravity framing, tests should be performed as described in Section 4.4.5.2 to evaluate the two-way punching shear strength of slab-column joints. For ordinary and intermediate moment frames, tests should be performed as described in Section 4.9.4.5 to evaluate the two-way shear strength of slab-column joints under cyclic loading.

3.5 General Considerations for Analysis

3.5.1 Flexural Stiffness

In structural analysis, selection of the moment of inertia used to represent the stiffness of the cross-section is important. Although designers are permitted to perform a rigorous analysis to select these values, Section 6.6.3 in ACI 318-14 provides values for moment of inertia that are considered appropriate. These values are based on research by MacGregor and Hage (1977), and were selected from results of frame tests and analyses. In general, recommended moments of inertia are presented as a fraction of the gross moment of inertia, I_g, for various members. These moments of inertia reflect a number of variables, including the amount of reinforcement, the extent of cracking, the variation of cracking along the length of the member, axial loads, creep, and inelastic behavior of steel and concrete. For this reason, different values are presented for factored loads and service loads (in Sections 6.6.3.1 and 6.6.3.2 of ACI 318-14). It is important that the flexural stiffness of the member appropriately reflects the overall behavior of the structure considering gravity and lateral loads. Studies on this have been carried out by Korinda (1972) and Hage (1974).

With the use of high-strength reinforcement, there is a potential that increased flexural cracking will occur at a given section, and that the extent of cracking over the length of the member will increase. An increase in cracking will result in a reduction in stiffness, which suggests that modification of the moments of inertia may be needed. It is anticipated that larger modifications will be needed as the grade of the reinforcement is increased.

Moment of inertia is also used in the consideration of second-order effects. While general analysis of a structure considers average moments of inertia across the structure, values used for consideration of second-order effects depends on the forces being calculated. The behavior of the structure is controlled by sidesway under lateral loads, for which the calculation of second-order end moments in columns depends on the average stiffness of the structure. However, for calculation of second-order moments within a column (i.e., between the ends of the column), the stiffness of the specific column being investigated is needed. For this reason, lower values of stiffness, *EI*, are used (Section 6.4.4.4 of ACI 318-14).

Increased cracking is not expected in columns reinforced with high-strength reinforcement. Therefore, effective *EI* values should be similar. However, it is possible that the amount of reinforcement used in columns could be reduced, or column sizes may be decreased. Such changes would be expected to have a more significant influence on stiffness than changes in cracking.

Recommendations. An analytical study as described in Section 4.8.1 should be performed to investigate the influence of high-strength reinforcement on flexural stiffness, *EI*.

An analytical study and confirming experimental tests as described in Section 4.8.2 should be performed to investigate the influence of high-strength reinforcement on the effective stiffness used estimate column second-order effects.

3.5.2 Moment Redistribution

Moment redistribution often provides for reserve capacity in members (or structures) in the event of overload. ACI 318 allows for redistribution of moments calculated by elastic analysis. The permissible amount of redistribution depends on the tensile strain of the longitudinal reinforcement at the extreme layer, with the maximum amount being 20%. Because moment redistribution depends on adequate ductility in the plastic hinge regions, adequate ductility must be provided. Using the ACI 318 approach, ductility is measured through the tensile strain achieved at ultimate, and a minimum strain of 0.0075, which is approximately 3.6 times the yield strain of Grade 60 reinforcement, is required. The commentary to ACI 318-14 states that the permissible redistribution is based upon analysis of flexural members with small rotation capacities “using conservative values of limiting concrete strains and lengths of plastic hinges derived from extensive tests.” Figure 3-9 shows ACI 318-14 permissible redistribution in terms of percent change in moment versus net tensile strain. As shown in the figure, the current permissible redistribution is conservative for yield strengths up to 80 ksi.

With the use of high-strength reinforcement, the yield strain of the reinforcement will increase, and the curves shown in the figure will become closer to the permissible redistribution line. For example, if Grade 100 is used, maintaining the minimum strain as 0.0075 will produce a reduction in the ratio of 2.2 times yield. Depending on the stress-strain response of high-strength reinforcement, this reduced ratio may not be sufficient to adequately provide for redistribution of moments. As a result, moment redistribution may not be allowed for high-strength reinforcement, or a modification to the permissible amount of moment redistribution may be needed.

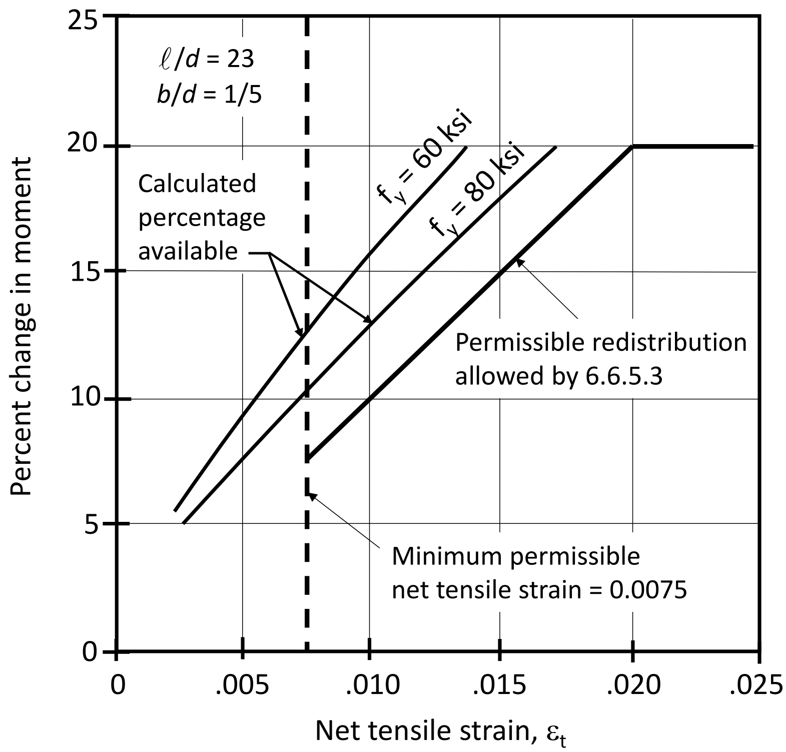


Figure 3-9 Permissible redistribution in accordance with ACI 318 (ACI, 2014a).

Recommendations. An analytical study as described in Section 4.8.3 should be performed to investigate the ability of structures reinforced with high-strength reinforcement to redistribute moments.

3.6 Seismic-Force-Resisting Systems

The response of reinforced concrete seismic-force-resisting systems to earthquake ground motion depends on many factors, including the configuration and geometry of the system, response of the components that comprise the system, the mechanical properties of the reinforcement and the concrete, the rotational capacities of the member cross-sections, the relative strengths in shear and flexure, and the detailing of longitudinal and transverse reinforcement.

Acceptability of the use of high-strength reinforcement in seismic-force-resisting systems will involve demonstration of the capability of such systems to provide a level of safety that is consistent with the seismic performance intent of ASCE/SEI 7-10, which has been quantified in FEMA P-695, *Quantification of Building Seismic Performance Factors* (FEMA, 2009) as 10% probability of collapse, on average, given maximum considered earthquake shaking. This can be evaluated in multiple ways. One approach is to establish response that is equivalent with the performance of seismic-force-resisting systems that have traditionally been designed and constructed using ASTM A706 Grade 60 reinforcement. A second approach is to directly evaluate and compare the collapse response of systems reinforced with high-strength reinforcement to the collapse performance criteria. However, there are some limitations in the ability of current modelling approaches to analytically simulate the response and behavior of certain reinforced concrete elements and failure modes. A third approach is to perform large scale experimental testing.

In determining whether equivalent performance is provided, both system and member performance must be assessed. Differences in member stiffness, energy dissipation, and backbone curves will likely result in differences in overall performance of systems reinforced with high-strength reinforcement. Differences in overall system performance should be used to determine if members reinforced with high-strength reinforcement require different performance capabilities (i.e., do they need larger rotational capacities?).

The use of high-strength reinforcement in seismic systems is investigated in detail in NIST GCR 14-917-30. ACI 318-14 limits reinforcement used in special seismic systems to Grade 60. NIST GCR 14-917-30 provides recommendations for code changes intended to allow the use of Grade 80 reinforcement in special seismic systems. It also identifies the research likely needed, and types of code changes likely required, for use of Grade 100 and Grade 120 reinforcement in special seismic systems.

Issues and concerns associated with high-strength reinforcement in seismic-force-resisting systems are identified and summarized in the sections that follow. Issues and concerns identified in this *Roadmap* are discussed in greater detail in NIST GCR 14-917-30.

3.6.1 Shape of Stress Strain-Relationship

The shape of the reinforcement stress-strain curve, and response to cyclic loading, affects several factors that influence the response of components in seismic-force-resisting systems. This was partially explored in NIST GCR 14-917-30. Using three different types of stress-strain curves (introduced in Section 3.1.1 and shown in Figure 3-1), moment-curvature relationships were computed for beams, columns, and shear walls for A706 Grade 60 and Grade 80 reinforcement, and Grade 100 reinforcement. Additional information about those curves is provided in Table 3-3. Stress-strain parameters used in this study were established based on a few actual stress-strain curves, so they are not considered representative of the full potential variation that is likely to exist in high-strength reinforcement. Usable elongations, ratios of tensile-to-yield strengths, and lengths of the yield plateau are likely to vary substantially from the values considered in NIST GCR 14-917-30.

Additional information regarding stress-strain curves for Grade 80 and Grade 100 reinforcement will soon be available. Additional moment-curvature relationships could be computed for stress-strain curves that more closely represent the properties of high-strength reinforcement that will be available in the United States. Moment-curvature relationships should be computed for stress-strain curves that represent the minimum properties likely to be required in ASTM standard specifications, as well as for average and maximum properties likely to be present in actual reinforcing bars.

Table 3-3 Stress-Strain Parameters for Reinforcement Used to Create Moment-Curvature Relationships in NIST GCR 14-917-30

Reinf. Type(1)	f_y (2)	ε_{sh} (3)	ε_{su} (4)	f_u / f_y (5)
100-S1	100 ksi	-	0.06	1.5
100-S2	100 ksi	0.00207	0.06	1.2
100-S3	100 ksi	0.0134	0.06	1.5
80-S3	80 ksi	0.0128	0.06	1.5
60-S3	60 ksi	0.0121	0.06	1.5

Notes: (1) Reinforcement type designations refer to Figure 3-1.
(2) Yield strength of reinforcement based on the 0.2% offset method.
(3) Strain defining the onset of strain hardening.
(4) Maximum usable strain. For the curves in Figure 3-1, maximum usable strain coincides with the tensile strength. A value of 6% is a representative lower-bound strain associated with the peak stress (tensile strength). In contrast, the strain associated with fracture elongation may be up to two times the strain associated with peak stress.
(5) Ratio of tensile-to-yield strength.

The shape of the stress-strain relationship is also likely to affect the spread of plasticity and the contribution of bar slip in joints and foundations to rotational capacity of members. In NIST GCR 14-917-30, rotational capacities were computed assuming that no bar slip occurred in joints and foundations, the usable elongation was 6%, and the plastic hinge length was equal to one-half of the overall member depth. Ignoring bar slip is conservative. Future analyses could include estimation of the effects of bar slip, but the variation in slip should account for the possibility that high-strength reinforcement might have greater relative rib area and mechanical anchorage in joints that might reduce bar slip. A usable elongation of 6% may not be achievable for all high-strength reinforcement, so smaller values should be investigated. The plastic hinge length will depend on the shape of the stress-strain relationship and member proportions, and can be different in each case.

Analytical models alone are not capable of accurately assessing the contribution of bar slip, the spread of yielding, and the overall influence of the shape of the stress-strain curve and cyclic loading behavior on rotational capacity. Tests are required to fully explore behavior, and correlation between analytical results and test data is likely needed.

A series of beam tests that explored the effect of tensile-strength-to-yield-strength ratio on cyclic behavior was performed in Japan as part of the New RC Project (Aoyama, 2001). Figure 3-10 shows a representation of the test specimens, and Figure 3-11 shows the two load-deflection curves that appear in the report. The results of these tests are only briefly discussed in Aoyama (2001) and a reference is not provided to the original source of the work. However, the results clearly demonstrate the importance of this ratio on how strength is maintained to drift levels of 5% or more.

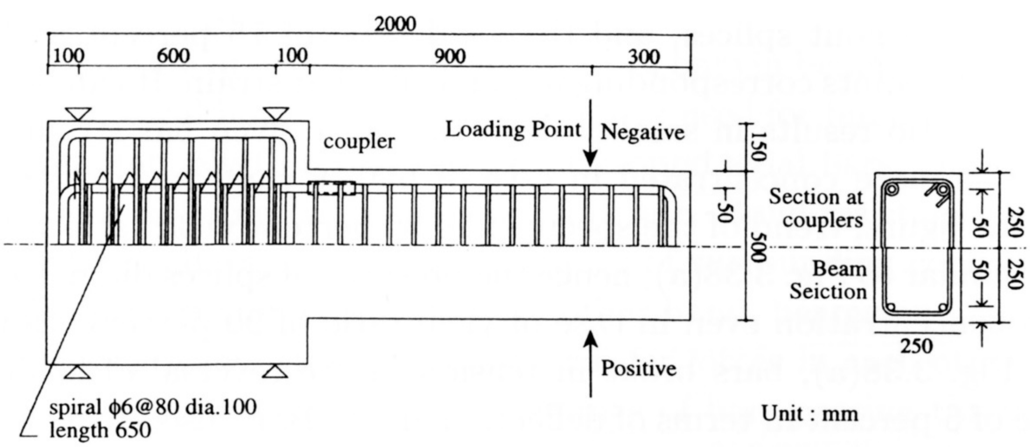


Figure 3-10 Beam specimens used to study the effect of the tensile-to-yield-strength ratio (Aoyama, 2001).

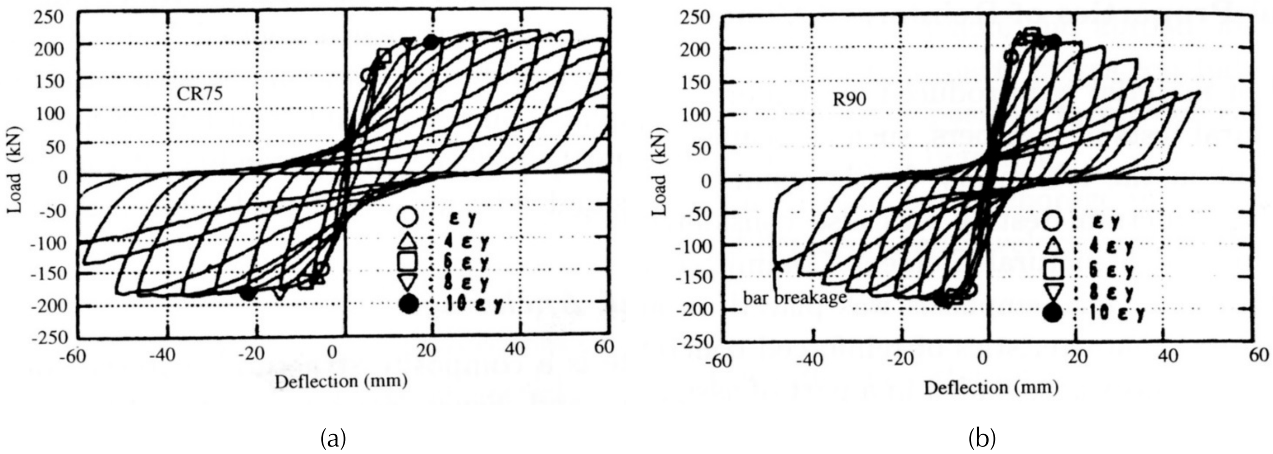


Figure 3-11 Load-deflection curves for beam specimens with: (a) yield ratio of 75% with splice; and (b) yield ratio of 90% without splice. Yield ratio is the inverse of the tensile-to-yield-strength ratio (Aoyama, 2001).

Tests that explore the effects of various mechanical properties on response are needed to establish required mechanical properties that should appear in ASTM standard specifications for high-strength reinforcing bars.

Recommendations. Analytical studies to investigate the effect of stress-strain properties on moment-curvature and rotational response should be performed as described in Section 4.9.1 and Section 4.9.2.

Experimental testing as described in Section 4.9.3 should be performed to help define required mechanical properties for high-strength reinforcement in seismic applications.

3.6.2 Intermediate Moment Frames

Provisions for intermediate moment frames were introduced in the 1983 edition of ACI 318. This edition also included provisions for slab-column, intermediate moment frames. The provisions for detailing typical beams and columns were based primarily on the consensus opinion of ACI committee 318. The spacing of hoops in the end regions of beams and columns has not changed since 1983. The 2002 edition of ACI 318 introduced specific requirements for punching shear, including a limit of $0.4\phi V_c$ on two-way shear caused by gravity loads. The 2011 edition of ACI 318 increased the design shear in columns from two times the shear determined from a force-based analysis, to a capacity-based shear associated with development of the nominal flexural strength at each end of the column, or an overstrength factor, Ω_o, times the shear determined from code level forces. For intermediate moment frames, the spacing of hoops, the maximum permissible gravity shear, and the design shear strength of columns and beams should be explored for members using high-strength reinforcement.

Beams and Columns

Studies have investigated the specific provisions of intermediate frames (Hwang and Hsu, 1993; Hwang and Hsu, 1994; Panahshahi and Lu, 1997; Sheth, 2003). Cyclic tests have been performed on columns with Grade 60 longitudinal reinforcement and transverse reinforcement similar to that required for intermediate moment frames (Han and Jee, 2005). Cyclic tests of beams and columns reinforced with high-strength longitudinal reinforcement, and with transverse reinforcement consistent with that required for intermediate moment frames, are needed to support the use of high-strength reinforcement.

Slab-Column Frames

Adequate performance of slab-column moment frames requires that a two-way shear failure not occur. The limit of $0.4\phi V_c$ for two-way shear caused by gravity loads is based on tests with Grade 60 longitudinal slab reinforcement (Pan and Moehle, 1989). Restricting gravity shear demands to this value is intended to provide protection against shear failure at story drifts up to 1.5%. Although these provisions appear independent of reinforcement ratio and bar strain, two-way shear strength depends on both, as described in Section 3.1.2.1.2. Use of high-strength slab reinforcement will likely reduce the reinforcing ratio, and increase strains in the reinforcement. Testing is needed to confirm that the current limit of $0.4\phi V_c$ provides adequate protection against two-way shear failure in slabs with high-strength reinforcement subjected to story drifts expected under maximum considered earthquake shaking. Nonlinear response history analyses should be performed to establish the expected drifts of intermediate slab-column moment frames with high-strength reinforcement.

Collapse Probability

An analytical study performed on one beam-column archetype building indicated that the collapse probability of intermediate moment frames was sufficiently low (Richard et al., 2010). However, comprehensive collapse investigation of intermediate moment frames with high-strength reinforcement is likely needed to support a code change for the use of high-strength reinforcement in intermediate seismic systems.

Recommendations. Experimental testing on components of beam-column and slab-column intermediate moment frames with high-strength reinforcement should be performed. Testing for intermediate moment frames is included as part of the testing described for special moment frames in Sections 4.9.4.1, 4.9.4.2, and 4.9.4.5. Analytical studies to investigate the collapse performance of intermediate moment frames should be performed as described in Section 4.9.5.

3.6.3 Special Moment Frames

The performance of special moment frames depends on the strength and deformation capacities of the members, the relative strengths between members, and the strength and detailing of joints at the intersection of members. Strength and deformation capacity of beams, columns, and joints are addressed in the sections that follow. The relative strength of beams and columns to achieve strong column-weak beam behavior is discussed.

3.6.3.1 Beams

Available cyclic load testing on beams with high-strength reinforcement is summarized in NIST GCR 14-917-30. Research available in English includes Sugano et al. (1990) and Kimura et al. (1993), both of which focus on the cyclic response of concrete beams. In Sugano et al. (1990), two of the eight beams tested as part of beam-column subassemblages had longitudinal and transverse reinforcement with yield strengths of 85 ksi and 125 ksi, respectively. The beam cross-section was 12 inches wide by 16 inches deep, with a shear span to effective depth ratio of about 3.5, and concrete compressive strength of 12,000 psi. In Kimura et al. (1993), 14 cantilever beams were tested, seven of which had longitudinal and transverse reinforcement with a yield strength of 115 ksi. Each specimen was 8 inches wide by 12 inches deep, with a shear span to effective depth ratio of 4.7, and concrete compressive strengths of 5,500 psi or 11,000 psi. In both studies, the beams reached drift ratios of 5%, with no more that 20% loss of peak strength.

Tavallali (2011) performed cyclic tests on beams with high-strength reinforcing bars. The tensile strength and total elongation of the bars were 98 ksi and 16%, respectively, for Grade 60 bars, and 117 ksi and 10%, respectively, for Grade 97 bars. Experimental data for two beam specimens, CC4-X and UC4-X, are presented here. The specimens consisted of two identical beams connected to a central stub through which load was applied, subjecting the specimens to single curvature bending. All transverse reinforcing bars were Grade 60, and the nominal compressive strength of the concrete was 6,000 psi. The layout of the longitudinal reinforcement was symmetrical, with identical top and bottom layers. The amount of longitudinal reinforcement was chosen so that both specimens reached nearly identical flexural strength, while limiting the shear stress, V/bd, to values approaching $6\sqrt{f_c'}$ (psi). The flexural reinforcement ratio, ρ, was 1.9% for the Grade 60 specimen and 1.4% for the Grade 97 specimen. The geometry and reinforcement details for these specimens are shown in Figure 3-12.

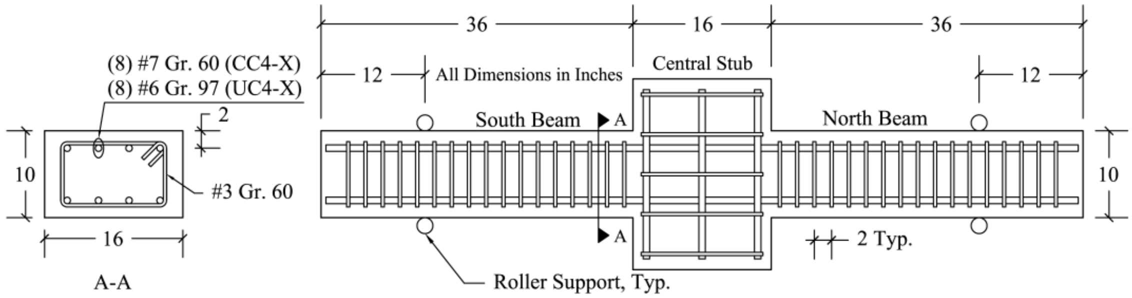


Figure 3-12 Details for beam specimens with high-strength reinforcing bars (adapted from Tavallali, 2011).

The measured shear-drift response for CC4-X is shown in Figure 3-13a, excluding the final monotonic loading event. The north beam tolerated two cycles of 5% drift while maintaining a load-carrying capacity similar to the peak load resisted in previous cycles.

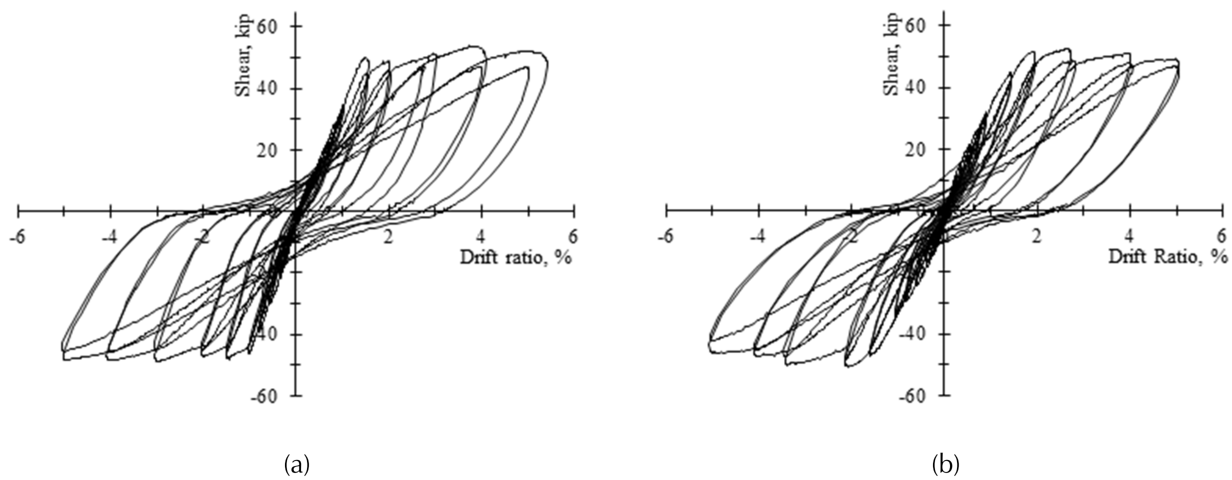


Figure 3-13 Measured shear versus drift ratio in beam tests: (a) Specimen CC4-X, with Grade 60 reinforcing bars; and (b) Specimen UC4-X, with Grade 97 reinforcing bars (Tavallali, 2011).

Specimen UC4-X had similar properties to Specimen CC4-X, with the exception that it was reinforced with Grade 97 (SAS 670) longitudinal bars. Figure 3-13b shows its measured shear-drift response. The Grade 60 bars were characterized by a tensile-to-yield strength ratio of 1.5, whereas the Grade 97 bars had a tensile-to-yield strength ratio of 1.2.

Compared with Specimen CC4-X, Specimen UC4-X demonstrated reduced post-cracking stiffness, increased yield deformation, and slightly narrower hysteresis loops. Results indicated that replacing Grade 60 longitudinal reinforcement with Grade 97 reinforcement, reduced in proportion to the yield strength of the reinforcement, led to comparable flexural strength and deformation capacity. These findings were corroborated by Pfund (2012) using similar beam specimens reinforced with ASTM A1035 Grade 120 longitudinal reinforcement, and suggest that beams reinforced with high-strength reinforcement are a viable option in seismic systems.

Recommendations. Additional experimental testing on beams with high-strength reinforcement should be performed as described in Section 4.9.4.1. Analytical studies to investigate the adequacy of the rotational capacity of beams with high-strength reinforcement should be performed as part of the analyses described in Section 4.9.5.

3.6.3.2 Columns

Available cyclic load testing on columns with high-strength reinforcement performed in the United States are summarized in NIST GCR 14-917-30. Restrepo et al. (2006) reports the testing of two circular columns that were scale models for the Oakland approach of the replacement San Francisco-Oakland Bay Bridge in California. Both column specimens were 3 feet in diameter and 9’-6” tall. One (Unit 1) was constructed using ASTM A706 Grade 60 longitudinal and transverse reinforcement, and the other (Unit 2) incorporated ASTM A1035 Grade 100 longitudinal and transverse reinforcement.

Unit 1 was constructed with two cages, each containing 42 No. 5 bars tied to No. 3 fuse-welded hoops at 1.56 inch spacing. The longitudinal reinforcement ratio, ρ_ℓ, was 2.54%, and the volumetric transverse reinforcement ratio, ρ_s, was 1.74%. The measured concrete compressive strength was 9,300 psi. Unit 2 was constructed with a single cage with the same reinforcing details. The measured concrete compressive strength was 8,200 psi. Both units were tested with an axial load of $0.07 f'_c A_g$, and subjected to reverse cyclic lateral loading. Figure 3-14 shows the hysteretic response of the two columns.

Unit 1 was cycled to drift ratios greater than 6%, when yielding of the hoops led to longitudinal bar buckling, followed by fracture. Unit 2 failed at 3.9% drift on the first cycle to a target drift of 6% via hoop fracture in the heat-affected region adjacent to the fuse weld. For the levels of story drift normally anticipated in special moment frame structures, the two columns exhibited effectively identical behavior.

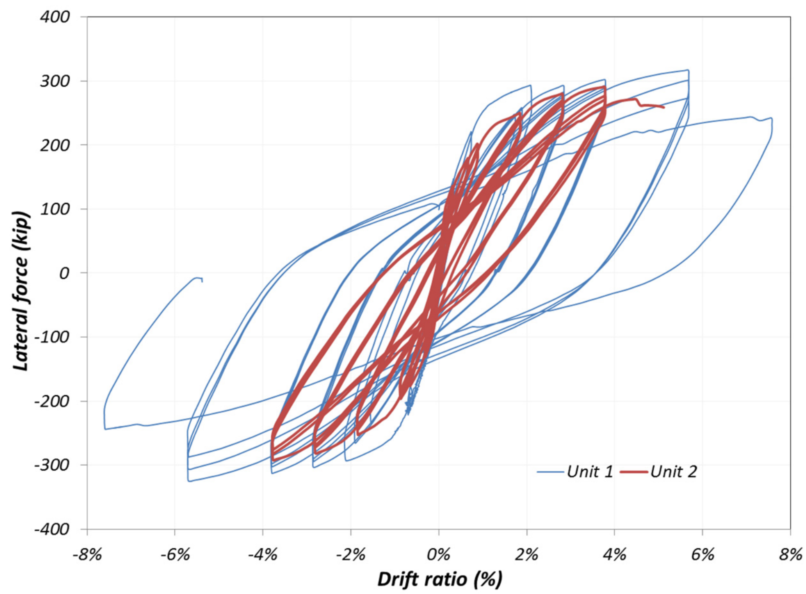


Figure 3-14 Hysteretic response of two circular columns based on Restrepo et al. (2006): Unit 1 including Grade 60 reinforcement, and Unit 2 including Grade 100 reinforcement (courtesy of J. Restrepo).

Rautenberg (2011) tested columns with high-strength reinforcing bars. Experimental data for two column specimens, CC-3.3-20 and UC-1.6-20, are presented here. Specimen CC-3.3-20 was designed with Grade 60 reinforcement in accordance with ACI 318-08 requirements for columns in special moment frames. Specimen UC-1.6-20 was similar, except that it was reinforced with ASTM A1035 Grade 120 longitudinal bars, adjusted to achieve nearly the same $\rho_\ell f_y$. The yield strength, tensile strength, tensile-to-yield strength ratio, and total elongation were 64 ksi, 92 ksi, 1.44, and 20% for the Grade 60 bars, and 133 ksi, 168 ksi, 1.26, and 8.6%, respectively, for the Grade 120 bars. The nominal concrete compressive strength was 6,000 psi. All transverse reinforcement was Grade 60. The test setup and loading protocol were similar to those used in the beam tests by Tavallali (2011). Both specimens were subjected a constant axial load of $0.2 f_c' A_g$. The amounts of longitudinal reinforcement was chosen so that the columns would reach nearly identical flexural strengths but limit the average shear stress to approximately $8\sqrt{f_c'}$ (psi). The typical geometry and rein′orcement details of these test specimens are shown in Figure 3-15.

The measured shear drift response for CC-3.3-20 is shown in Figure 3-16a. The controlling column completed the first cycle to 5% drift, but the longitudinal bars buckled during the second cycle at that drift ratio. The measured shear-drift response of UC-1.6-20 is shown in Figure 3-16b. The controlling column of Specimen UC-1.6-20 completed the first half-cycle to

5% drift, but the longitudinal bars buckled during the second half-cycle at that drift ratio. Similar findings were obtained by Tretiakova (2013) on cyclic tests of concrete columns reinforced with SAS 670 Grade 97 steel bars. The columns tested in Tretiakova (2013) were nearly identical to those tested in Rautenberg (2011).

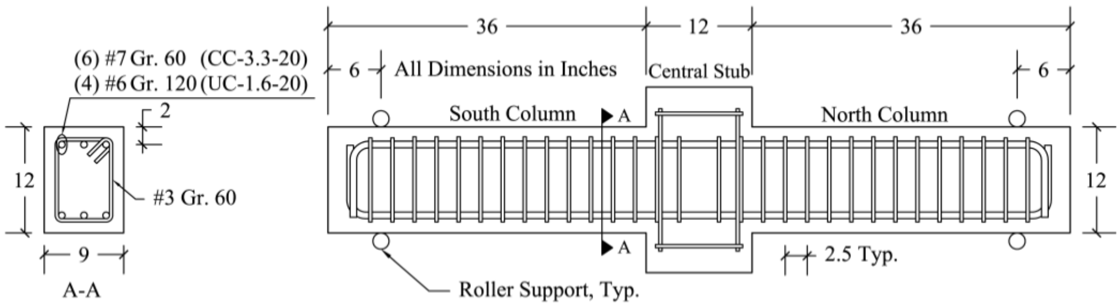
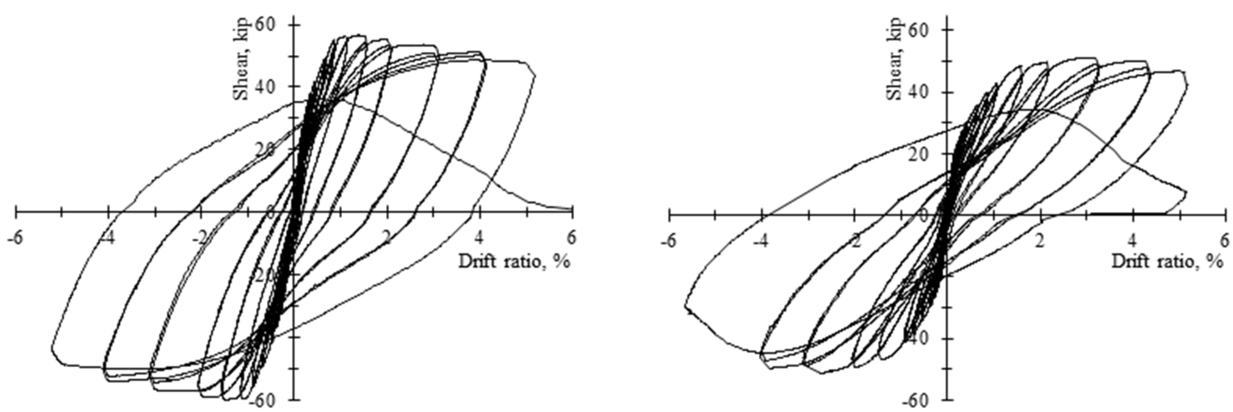


Figure 3-15 Details for column specimens with high-strength reinforcing bars (adapted from Rautenberg, 2011).



(a) (b)

Figure 3-16 Measured shear versus drift ratio in column tests: (a) Specimen CC-3.3-20, with Grade 60 reinforcement; and (b) Specimen UC-1.6-20, with Grade 120 reinforcement (Rautenberg, 2011).

Many tests have been conducted on columns with high-strength reinforcement in Japan. Data from tests on 115 column specimens has been used to calibrate statistical models for predicting ultimate deformation capacity (Ishikawa et al., 2008).

Recommendations. Additional experimental testing on columns with high-strength reinforcement should be performed as described in Section 4.9.4.2. Analytical studies to investigate the adequacy of the rotational capacity of columns with high-strength reinforcement should be performed as part of the analyses described in Section 4.9.5.

3.6.3.3 Shear Demand on Beams and Columns

Shear demand on beams and columns in special moment frames is based, in part, on the flexural strength of the members. Commentary Section R18.6.5 in ACI 318-14 notes that shear demand should be calculated using a reinforcement stress of at least $1.25f_y$ "because the actual yield strength of the longitudinal reinforcement may exceed the specified yield strength, and because strain hardening of the reinforcement is likely to take place at a joint subjected to large rotations." The presence of slabs will also increase flexural strength over the nominal strength of the rectangular beam cross-section alone. If the ratios of expected-to-specified yield strength, expected tensile-to-yield strength, and measured tensile-to-yield strength are substantially greater for high-strength reinforcement, the factor of 1.25 may need to be increased.

At the time that $1.25f_y$ was established as the basis for probable moment strength, buildings typically had more frames participating in seismic force resistance, the size of the members in frames was smaller. Part of the reason why the 1.25 factor was not larger was because spalling of concrete cover was more significant in smaller members and reduced the strength of the member more significantly. In current practice, buildings are generally designed with fewer seismic frames and larger members. Spalling of cover concrete results in less of a reduction in overstrength, because the cover is a smaller percentage of the member depth. For this reason, the acceptability of the 1.25 factor should be reassessed Grade 60 reinforcement, and investigated for high-strength reinforcement.

Recommendations. Analytical studies to investigate moment-curvature response and shear demands on beams and columns should be performed as described in Section 4.9.1. An analytical study to investigate the 1.25 factor for probable moment strength used in calculating shear demands should be performed as described in Section 4.9.2.

3.6.3.4 Strong Column-Weak Beam Behavior

Section 18.7.3.2 of AC1 318-14 includes provisions for strong column-weak beam behavior. It requires that the sum of the nominal flexural strengths of columns framing into a joint be greater than six-fifths times the sum of the nominal flexural strengths of beams framing into the joint. The required ratio was originally determined largely through engineering judgment and the results of tests of hinging regions in beams, at a time when the yield strength of flexural reinforcement used in moment frames was 60 ksi.

The purpose of the strong column-weak beam requirement is to promote yielding and formation of plastic hinges in beams before yielding and hinging occurs in columns. Hinging in beams spreads story drifts more uniformly over the height of the structure, and helps prevent the formation of story collapse mechanisms caused by hinging in columns. The requirement was also meant to protect joints. Joint integrity is weakened by yielding of the bars of members framing into the joint. If both the column and beam bars are yielding, joint stability is diminished.

FEMA P-695 (FEMA, 2009) indicates that the current strong column-weak beam requirements do not preclude column hinging in concrete structures. Although the provisions protect against the formation of a single-story mechanism, multi-story collapse mechanisms can form, which include only a fraction of the total number of stories in a building. Eliminating the potential for multi-story mechanisms, and all column hinging is unrealistic and unnecessary. However, collapse mechanisms that do form need to spread yielding over enough stories so that collapse resistance meets the performance intent of ASCE/SEI 7-10.

The effect of high-strength reinforcement on strong column-weak beam provisions in ACI 318-14 should be investigated because the mechanical properties of high-strength reinforcement may be less favorable to the spread of yielding throughout the structure. In particular, the tensile-to-yield strength ratio is likely to be less than Grade 60 reinforcement. Nonlinear response history analyses could be used to explore whether the existing six-fifths ratio is at least as effective for high-strength reinforcement as it is for Grade 60 reinforcement.

Recommendations. An analytical study to investigate strong column-weak beam requirements considering high-strength reinforcement should be performed as part of the analyses described in Section 4.9.5. Ideally, analytical models should be calibrated with results of beam, column, and joint cyclic load tests in Section 4.9.4.

3.6.3.5 Beam-Column Joints

The size of a beam-column joint must be sufficient to transfer forces from yielding reinforcement to the concrete through bond. Current rules and guidance on joint depth are empirical, and based on tests of conventionally reinforced components and subassemblies of frames. High-strength reinforcement will develop higher forces for a given bar diameter, and deeper joints may be required if the bond stresses that can be developed are limited by the tensile strength of the concrete. Bond stresses should be investigated for both high-strength beam reinforcement and high-strength column reinforcement passing through beam-column joints.

3.6.3.5.1 Depth of Interior Joints

Section 18.8.2.3 of ACI 318-14 requires the depth of interior beam-column joints of normal weight concrete to be 20 times the diameter of the largest beam bar ($20d_b$) passing through the joint. Considering that a bar may be yielding in tension on one side of the joint and may be near zero stress (or in slight compression) on the other, this might be considered a relatively shallow depth. The intent of ACI 318 joint requirements is to provide acceptable overall performance of a building through reasonable (but not total) control of bar slip through the joint. In joints tested by Zhu and Jirsa (1983) and Lin et al. (2000), bond failure of longitudinal beam bars occurred at drift levels comparable to those expected for maximum considered earthquake ground motions.

The required joint depth for use with high-strength reinforcement was investigated in NIST GCR 14-917-30. Equations of varying complexity are available to account for the major factors that influence joint depth, including bar yield strength, top bar effects, column axial load, and concrete compressive strength. For Grade 80 and Grade 100 reinforcement, minimum joint depths of $26d_b$ and $35d_b$, respectively, were recommended.

Thirteen tests of interior beam-column joints were performed as part of Japan's New RC Project (Aoyama, 2001), and an equation for joint depth was developed. Similar factors are considered, but they are treated differently enough that further study is warranted.

Recommendations. Available tests of interior beam-column joints should be assembled and reviewed. Additional interior joint tests should be performed as described in Section 4.9.4.3 to confirm joint depth recommendations available in the literature or to make new recommendations on required joint depth.

3.6.3.5.2 Depth of Exterior Joints

At exterior beam-column joints, beam longitudinal bars are usually terminated with 90-degree hooks or headed bars located within the joint. The joint depth must be sufficient to develop the beam bars within the joint, and the hooks or heads must terminate near the far side of the joint so that a diagonal strut can develop to resist joint shear. Because high-strength reinforcement will develop higher forces for a given bar diameter, there is a potential concern for increased bar slip and concrete crushing due to larger local contact stresses around hooked bar bends or under heads of headed reinforcement.

In Japan, a concrete compressive strength of at least 11 ksi is required for joints with Grade 100 longitudinal reinforcement that is terminated with 90-degree hooks (Aoyama, 2001; NIST, 2014). This requirement, however, is based on tests of beam-column joints with transverse reinforcement that is not in compliance with ACI 318, and typical drift levels in Japanese buildings (i.e., 1.5%) (Otani, 1991), are much less than drift levels expected in U.S. buildings (i.e., 2.5% to 4%).

ACI 318-14 restricts the use of headed deformed bars to Grade 60, requires that heads have a net bearing area of four times the area of the bar ($4A_b$), and requires the use of Class HA heads, in which the bar-to-head connection must develop the minimum specified tensile strength of the bar. There is interest in increasing the maximum grade for headed reinforcement to reduce congestion in exterior joints. Under high ductility demands, potential bar slip in the joint could result in additional demands on the heads of headed reinforcement. It is possible that increased demands on the heads of headed reinforcement could result in the need for larger bearing areas under the head and stronger bar-to-head connections. Heads with a net bearing area of up to nine times the area of the bar ($9A_b$) are available, and ASTM A970, *Standard Specification for Headed Steel Bars for Concrete Reinforcement* (ASTM, 2013), includes a Class B head that develops the specified elongation of the bar. It is possible that a new Class HB head may be needed in ASTM A970 Annex A1 to comply with other deformation obstructions and bearing face feature limitations.

No exterior joint tests with ACI 318-compliant joint detailing and Grade 80 or higher longitudinal reinforcement have been identified to date. However, as reported in NIST GCR 14-917-30, full-scale shake table testing of a five-story special moment frame (Chen et al., 2012) demonstrated good performance in exterior joints with Grade 120 beam reinforcement terminated with 90-degree hooks. The measured yield and ultimate tensile strength of the reinforcing bars were 130 ksi and 160 ksi, respectively, and the compressive strength of the concrete averaged 7.5 ksi. The building was subjected to input ground motions that caused rotation demands to exceed 0.06 radians in the beams. Some of the top and bottom bars in the beams fractured, but no distress was observed in the beam-column joints.

Although full-scale shake table tests include some promising results for exterior joints, controlled testing of cyclically loaded exterior joint subassemblies is needed.

Recommendations. Additional exterior joint tests should be performed as described in Section 4.9.4.4 to confirm the required depth of exterior joints with high-strength hooked or headed bars.

3.6.3.5.3 Shear Reinforcement in Exterior Joints

The potential for splitting failure in a joint was identified by Marques and Jirsa (1975). Because of higher forces associated with high-strength reinforcement, exterior beam-column joints with high-strength beam and column reinforcement have an increased potential for splitting.

Full-scale shake table tests reported by Chen et al. (2012) included lightly reinforced exterior joints with Grade 120 longitudinal reinforcement that did not exhibit joint splitting. Exterior joints with more closely spaced bars, however, are expected to develop large splitting forces, so additional study is needed.

Recommendations. Additional exterior joint tests should be performed as described in Section 4.9.4.4 to confirm requirements for joint transverse reinforcement when high-strength longitudinal reinforcement is used.

3.6.4 Flexure-Critical Special Structural Walls

ACI 318 provisions for special structural walls include requirements for shear strength, axial-moment (P-M) interaction, boundary elements, coupling beams, and wall piers. These provisions are intended to protect against brittle or limited-ductility states, and to encourage energy-dissipating, ductile mechanisms. Other than for confinement, all reinforcement in special structural walls is limited to Grade 60.

Slender structural walls with height-to-length aspect ratios of three and larger are flexure-critical, but for walls specifically designed and detailed for flexural behavior, this ratio can be as low as two. Flexure-critical walls are expected to be ductile, which is possible if shear failure is prevented.

Design for axial-moment (P-M) interaction of vertical reinforcement in special structural walls follows provisions applicable to other compression members. However, the end regions of special structural walls are required to have transverse reinforcement to restrain against buckling of vertical bars and to provide compression strain ductility similar to that required for special moment frame columns.

Diagonally reinforced coupling beams are often required in special structural walls, and are highly ductile if confined in accordance with ACI 318-14. These beams are difficult to construct due to congestion of the reinforcement.

Wall piers are vertical segments of a shear wall system with geometric aspect ratios that are similar to columns. As such, many of the requirements for special moment frame columns are applied for ductile behavior. Wall piers are often heavily reinforced.

Use of high-strength reinforcement has the potential to reduce the area of reinforcement required in special structural walls and reduce congestion in boundary elements and wall piers. Reinforcement congestion can limit the design strength that can be provided in coupling beams. Verification of the performance of high strength reinforcement in diagonally reinforced coupling beams, and for longitudinal bars and hoops in conventionally reinforced coupling beams, could allow for greater nominal shear strength for given coupling beam dimensions.

Available tests of structural walls with high-strength reinforcement are identified and summarized in NIST GCR 14-917-30. There are few tests that are directly applicable to slender wall design common in the United States. In tests performed by Kimura and Ishikawa (2008), only three specimens with Grade 100 reinforcement were similar in configuration to U.S. practice. These tests included three 1/5-scale, rectangular, slender wall specimens reinforced with SD685 bars (i.e., nominal yield strength of 100 ksi) in the boundary elements, and in the horizontal and vertical web reinforcement. The shear span ratio of these specimens was 2.0. Although the transverse reinforcement area did not comply with ACI 318 requirements for special boundary elements, the spacing and configuration did.

Hysteresis loops from the Kimura and Ishikawa (2008) tests showed no degradation in strength up to the ultimate drift. The ultimate drift was 1.5% for the wall with an axial load ratio $(P / A_g f_c')$ of 0.15, and 2% for the walls with an axial load ratio of 0.10. All walls exhibited a flexural compression failure mode in which the concrete crushed and the boundary bars buckled. The length of bar yielding at the base of the specimens was about half the wall length. The drift levels achieved in the specimens with Grade 100 reinforcement were on the low end of what is expected from walls with Grade 60 reinforcement, which is about 2%.

In Dazio et al. (2009), six walls with high-strength reinforcement and yield strength close to 80 ksi were tested. In the specimens, axial load, quantity of reinforcement, elongation, and transverse reinforcement at the ends of the walls varied. The transverse reinforcement in the boundary elements did not comply with the ACI 318 requirements, but the results demonstrated possible trends in how walls with high-strength reinforcement might respond in earthquakes:

- In one test, the need for ductile reinforcement, with adequate strain hardening was apparent. In this test, typical vertical bars with uniform elongation of 2.3% ruptured at 0.6% drift, and boundary element bars with uniform elongation of 4.6% ruptured at 1% drift. Rupture of these bars also appeared to be influenced by low reinforcing ratios (0.3% for typical vertical bars, and 1.3% for boundary element bars).
- Another test demonstrated that a ratio of transverse reinforcement spacing to longitudinal bar diameter, s/d_b, of 7.5 does not provide adequate resistance to buckling of longitudinal boundary element bars. All six longitudinal bars in a boundary element buckled at a drift of 1.4%, and then fractured during loading in the reverse direction.
- A pair of tests further highlighted the benefit of closer spaced hoops in boundary elements. Two wall specimens had the same vertical and horizontal wall reinforcement. One had a ratio of transverse reinforcement spacing to longitudinal bar diameter, s/d_b, of 6.3, with an axial load ratio, $P / A_g f_c'$, of 0.058. The other had an s/d_b, ratio of 4.2, with an axial load ratio of 0.11. Although the second wall specimen had nearly twice the axial load ratio, the observed deformation capacities were similar to the first wall because of the reduced hoop spacing. Onset of boundary element bar buckling occurred at a drift of 1.7% in both walls.
- A pair of tests demonstrated the need for a minimum amount of vertical reinforcement. Two wall specimens had low vertical reinforcement ratios (0.3% for typical vertical bars, and 1.3% for boundary element bars). In these specimens, a wide horizontal crack formed. Instead of experiencing distributed yielding, these bars fractured.

Lowes et al. (2012) tested four wall specimens, two of which had boundary element reinforcement with a yield strength of 85 ksi, and boundary element detailing in accordance with ACI 318-11. At the base of two specimens, the vertical web and boundary element reinforcement were spliced. In one specimen, bar buckling began during the second cycle to 1% drift, and the bars fractured below the splice during the second cycle to 1.5% drift. In the other specimen, the bars buckled above the splice at 1.05% drift during the loading cycle to 1.5% drift. These tests demonstrated that bar yielding can occur at both the top and bottom of a splice, and that strains can be concentrated at the base of the wall, resulting in bar fracture.

Recent studies by Luna et al. (2014) on low aspect ratio (squat) rectangular walls with Grade 60 reinforcement and normal-strength, normal-weight concrete have shown the effects of bidirectional loading to be important in terms of peak in-plane shear strength, in-plane stiffness and the deterioration of strength and stiffness with repeated cycling at large story drifts. The out-of-plane loadings, which included displacement and twisting, were not controlled. The inelastic cyclic response of rectangular and flanged walls under bidirectional horizontal loading is not well understood and a series of tests focused on the effect of controlled bidirectional loading histories is needed. Part of that testing program would address the use of high-strength reinforcement and its use to reduce bar size for a given spacing, or increase bar spacing for a given bar size.

Although the minimum specified yield strength of Grade 80 reinforcement is not appreciably greater than the yield strength of the Grade 60 reinforcement currently being produced, the expected yield strength of Grade 80 reinforcement will likely exceed 90 ksi, for which there are little or no data. As a result, the adequacy of walls reinforced with high-strength reinforcement cannot be verified based on available data. Tests of flexure-critical walls, including coupled walls, are likely needed to support a code change for the use of high-strength reinforcement in boundary elements, webs, and coupling beams in flexure-critical walls.

Recommendations. Additional experimental testing on flexure-critical walls with high-strength reinforcement should be performed as described in Section 4.9.10. Analytical studies to investigate the performance of flexure-critical walls with high-strength reinforcement should be performed as part of the analyses described in Section 4.9.11.

3.6.5 Shear-Critical Walls

The seismic response of low aspect ratio walls, also referred to as squat or shear-critical walls, is poorly understood, even for walls with conventional Grade 60 reinforcement. Reinforcement serves multiple purposes in squat reinforced concrete walls: (1) horizontal web reinforcement delays diagonal tension failure; (2) vertical web reinforcement anchors the diagonal compression struts; (3) vertical web reinforcement delays sliding at horizontal construction joints; (4) web reinforcement confines the diagonal compression struts; and (5) vertical boundary reinforcement anchors compression struts, improves the integrity of the struts at the toes of the wall, and controls crack widths at the wall boundaries (Luna et al., 2014).

The roles of reinforcement in squat walls will vary as a function of drift ratio, with roles changing at drift ratios greater than those associated with peak shearing strength. These roles are not understood for conventional reinforcement in walls with rectangular cross-sections and uniform distributions of web reinforcement. The hysteretic response of walls equipped with boundary columns or flanges, which are common in building construction, will be more complicated than for rectangular walls, given the number of additional cross-section and rebar variables, and the unknown effects of seismic loading perpendicular to the plane of the web (and in plane of flanges) that will introduce out-of-plane flexure and shear.

The impact of replacing conventional reinforcement with high strength reinforcement will depend, in part, on whether it is used to reduce bar size for a given spacing or increase bar spacing for a given bar size, and on the anticipated range of response. If the goal is to increase bar spacing, the impact could be significant.

Reverse cyclic testing of large-size squat reinforced concrete walls will be needed to characterize the effect of using high strength reinforcement. Walls with different aspect ratios, concrete strengths, reinforcement grades, and plan geometries (rectangular, barbell, and flanged) should be evaluated for the purpose of ensuring that the use of high strength reinforcement does not lead to poorer performance than walls with an equivalent shearing capacity (as measured by Chapter 18 in ACI 318-14) constructed with conventional reinforcement. Benchmark tests should first be performed on walls constructed with Grade 60 reinforcement and prescriptive details consistent with ordinary and special construction.

Although squat structural wall specimens with Grade 100 reinforcement have been tested in Japan (Aoyama, 2001; Kabeyasawa and Hiraishi, 1998), these walls had very thin webs and column-type boundary elements. Tests were performed to study: (1) the strength of walls that first yield in flexure but fail due to shear-compression (web crushing); (2) the impact of bi-directional loading; and (3) the shear strength of walls. The walls failed predominately due to concrete crushing in the web (a brittle mode) at relatively low drifts. Few squat walls are constructed in the United States with thin webs and barbells, so the Japanese test results cannot directly support a code change to permit the use of high strength reinforcement in shear-critical walls.

Recommendations. Additional experimental testing on shear-critical walls with high-strength reinforcement should be performed as described in Section 4.9.8. Analytical studies to investigate the performance of shear-critical walls with high-strength reinforcement should be performed as part of the analyses described in Section 4.9.11.

3.6.6 Diaphragms

Issues associated with use of high strength reinforcement in diaphragms are similar to those discussed for flexure-critical and shear-critical walls. High strains in the reinforcement of diaphragm collectors and chords can lead to excessive cracking of the diaphragm. Section 18.12.7.2 in ACI 318-14 recognizes this, and limits the stress from design earthquake forces to 60 ksi for bonded tendons, which are often used as diaphragm chord reinforcement. In NIST GCR 10-917-4, *Seismic Design of Cast-in-Place Concrete Diaphragms, Chords, and Collectors–A Guide for Practicing Engineers* NIST (2010a), the following statement is made:

> "Although stress in other collector and chord reinforcement is not limited, consideration should be given to deformation compatibility between tension chords, collectors, and the floor slab. High tensile stress and strains in collectors and chords can result in excessive cracking that will migrate into the slab."

However, given that deformation demands on diaphragms are likely smaller than those in the critical regions of shear walls, there is a basis for accepting Grade 80 reinforcement for diaphragms, especially recognizing that the expected yield strength of Grade 60 reinforcement is not appreciably less than 80 ksi. Testing of diaphragms is not considered to be a high priority, and testing is not recommended at this time. Data from tests on flexure-critical and shear-critical walls should be mined for the purpose of assessing implications for diaphragms, noting that: (1) tests of walls normally produce peak shear and bending moment at one location; and (2) diaphragms may experience peak shears and bending moments at different locations along the span.

Recommendations. No further experimental investigation or engineering study is recommended at this time. Requirements for use of high-strength reinforcement in diaphragms could be decided based on information from testing on flexure-critical and shear-critical walls and consensus opinion.

Chapter 4

Research Studies

In most cases, issues related to production and fabrication of high-strength reinforcement, and design and construction using high-strength reinforcement, require further study or research to resolve. This chapter outlines the experimental research and engineering studies needed to adequately investigate the use of high-strength reinforcement for use in general reinforced concrete design.

4.1 Objectives

The objectives of this research plan are to: (1) further investigate the use of high-strength reinforcement; and (2) support the development of code change proposals for ACI 318-14, *Building Code Requirements for Structural Concrete and Commentary* (ACI, 2014) to allow the general use of reinforcing steel bars in excess of Grade 60.

4.2 Overview

Studies are related to the issues identified and discussed in Chapter 2 and Chapter 3. They are intended to resolve the issues in sufficient detail to substantiate a code change, where recommended. Each research or study topic includes a description of: (1) the purpose for the work; (2) details of the study; (3) the recommended team; and (4) the anticipated timeline. Estimated budget requirements and recommendations for prioritization are provided in Chapter 5.

4.3 Bar Production and Specification

Studies related to bar production and specification are summarized in Table 4-1.

Table 4-1 Studies Related to Bar Production and Specification

Reference Section	*Research or Engineering Study*	*Estimated Timeline*
4.3.1	Mechanical Properties of Recent Heats of High-Strength Reinforcement	6 months
4.3.2	Detailed Mechanical Property Tests of Grade 100 and Grade 120 Reinforcement	18 months

4.3.1 Mechanical Properties of Recent Heats of High-Strength Reinforcement

Collection and presentation of mechanical properties of recently rolled heats of high-strength reinforcement.

Purpose. To determine the range of values for key mechanical properties of high-strength reinforcing bars that manufacturers are currently capable of producing. Results from this study are intended to help in identifying mechanical properties that need to be included as variables in future tests, as well as help in defining structurally acceptable properties for future development of ASTM standard specifications. Data of interest include the tensile-to-yield strength ratio and uniform elongation, as well as the effects of bend radius, bar deformations, and bar marks on bend tests.

Details of Study. In this study, data should be collected and analyzed with the assistance of the Concrete Reinforcing Steel Institute (CRSI) and its member producers to allow anonymity of the data, but also allow the collection and establishment of reinforcing bar properties for possible inclusion in future ASTM standard specifications. Collected data should include yield strength, length of yield plateau, tensile strength, tensile-to-yield strength ratio, uniform elongation (including the means by which elongation was measured), and fracture elongation, and should be presented in a series of charts and tables. In addition, bend tests should be performed on a representative sample of bars from various producers to gain insight on fracture potential, and bend-rebend tests should be performed to evaluate strain-aging characteristics.

Team. One researcher, one part-time graduate student, and one part-time laboratory technician to assist.

Timeline. Approximately 6 months.

4.3.2 Detailed Mechanical Property Tests of Grade 100 and Grade 120 Reinforcement

An experimental study including collection of available test data from producers, additional testing, and statistical analysis of mechanical property data from different heats of high-strength reinforcement.

Purpose. To obtain data from detailed tests of high-strength reinforcing bars to determine mechanical properties for use in specifying high-strength reinforcement in future ASTM standard specifications. The goal is to determine mechanical properties that are of primary interest to structural designers. This study is considered necessary to help ensure that high-strength reinforcing bars are produced with adequate strength and ductility to result in safe and serviceable designs.

Details of Study. This study consists of experimental testing of No. 4 to No. 18 bars (and larger bars, if available), to collect:

- complete stress-strain curves with values for modulus of elasticity, yield strength, length of yield plateau, tensile strength, uniform elongation, and total elongation (across fracture) measured during standard tensile tests;
- low-cycle fatigue characteristics at ambient and low temperature conditions measured during large strain amplitude cyclic tests;
- toughness of bars at various temperatures, and transition temperature, using Charpy V-notch tests;
- potential for fracture using bend tests; and
- strain-aging characteristics using bend-rebend tests.

Tests should be conducted on reinforcing bars from a minimum of three different producers and three different heats per producer. Other measurements of interest include bar deformations, chemical composition, and whether or not the fabrication (i.e., bending) tolerances are being met. Such measurements may be collected and analyzed with the assistance of the Concrete Reinforcing Steel Institute (CRSI) and its member producers. Resulting data should be presented in a series of charts and tables including results from statistical analyses.

Team. One researcher, one part-time graduate student, one part-time laboratory technician to assist, and one part-time engineering practitioner for advice and consultation.

Timeline. Approximately 18 months.

4.4 Strength of Members

Studies related to strength of members are summarized in Table 4-2.

4.4.1 Flexural Strength and Tensile Strain Limits

An analytical study in which moment-curvature relationships are developed for beams, columns, and walls using various stress-strain curve shapes and different mechanical properties for the reinforcing bars and the concrete.

Purpose. To investigate the effects of different shapes of stress-strain curves on moment-curvature relationships. Analytical studies are intended to: (1) evaluate compression-controlled and tension-controlled strain limits; and

Table 4-2 Studies Related to Strength of Members

Reference Section	*Research or Engineering Study*	*Estimated Timeline*
4.4.1	Flexural Strength and Tensile Strain Limits	18 months
4.4.2	Required Deflection of Flexural Members Subjected to Gravity Loads	2 years
4.4.3	Column Strength	3 months
4.4.4	Tension Regions of Shells and Folded Plates	3 years
4.4.5.1	One-Way Shear in Beams without Shear Reinforcement	3 years
4.4.5.2	Two-Way Shear in Slabs without Shear Reinforcement	3 years
4.4.6	Shear Strength of Beams with Shear Reinforcement	3 years
4.4.7	Shear Friction	3 years
4.4.8	High-Cycle Elastic Fatigue of High-Strength Reinforcing Bars	3 years

(2) evaluate whether traditional ACI 318 design assumptions for calculating flexural strength (i.e., equivalent stress block for concrete and elastic-plastic stress-strain curve for reinforcing steel) are appropriate when using high-strength reinforcement. Results are also intended for use in evaluating the maximum tensile strain in the reinforcement and its effects on the deformation capacity of the members to determine whether or not a maximum strain limit is required to avoid fracture of reinforcement prior to crushing of concrete.

Details of Study. This study consists of analytical investigations to develop moment-curvature relationships and evaluate the monotonic deformation capacity of members reinforced with high-strength reinforcement, including beams, columns, and walls. The study should include review of existing experimental data and comparison with applicable experimental results from tests recommended in Section 4.4.2. The following variables should be considered:

- Concrete compressive strength (varying from 4,000 psi to 12,000 psi)
- Confined and unconfined concrete
- Yield strength of reinforcement
- Reinforcement tensile-to-yield strength ratio
- Reinforcement post-yield stiffness
- Shear-span ratio
- Reinforcement ratio
- Axial load ratio

- Ratio of the confined core to overall cross-section dimensions

Recommendations regarding the compression-controlled and tension-controlled strain limits, as well as the need for a maximum tensile strain limit, should be provided. In addition, possible changes to the traditional design assumptions for calculating flexural strength should be provided, if necessary.

Team. One researcher and one part-time graduate student.

Timeline. Approximately 18 months.

4.4.2 Required Deflection of Flexural Members Subjected to Gravity Loads

Analytical studies and confirming experimental tests to evaluate the impact of the shape of different stress-strain curves on the load-deflection behavior of flexural members, and to determine if adequate deflection is provided when a beam reinforced with high-strength reinforcement is overloaded.

Purpose. To determine the load-deflection behavior of flexural members reinforced with high-strength reinforcement, and investigate deflection response prior to failure when overloaded. Analytical studies and tests should consider tensile-to-yield strength ratios for high-strength reinforcement that can be less than 1.25. Results are intended to provide information regarding the importance of the shape of the stress-strain curve, whether or not a yield plateau is necessary, and what uniform elongation is needed for gravity-loaded flexural members

Details of Analytical Study. This study consists of analytical investigations beginning with a literature search to determine if applicable tests are available for the range of mechanical properties being considered. Moment-curvature analyses and nonlinear beam deflection analyses should be developed for a range of beam and slab designs that are consistent with current practice. This study can be used to complement the studies outlined in Section 4.4.1. Initial analytical models should be calibrated to test results, which will assist in establishing the level of detail needed for beam modeling. The following variables should be considered:

- Member depth (e.g., ranging from one to two times minimum depth)
- Yield strength of reinforcement (e.g., up to Grade 120)
- Reinforcement stress-strain curves of different shapes, with varying tensile-to-yield strength ratios, uniform elongation, and length of yield plateau

Details of Experimental Study. This study consists of experimental testing designed around confirming the results of the analytical studies. The number and range of specimens should target the limits of the mechanical properties and section properties that were studied analytically. Tests should attempt to track bar elongations as the beam is loaded to failure, and should also explore the net tensile strain needed for various stress-strain curve shapes. In addition, the tests should be used to track deflections at service load levels and, if possible, to explore effectiveness of crack control requirements. For budgeting purposes, the number of test specimens is anticipated to be on the order of 10.

Team. One researcher, two full-time graduate students (two years), and one part-time laboratory technician to assist.

Timeline. Approximately 2 years.

4.4.3 Column Strength

An analytical study in which moment-curvature relationships are developed for columns using various stress-strain curve shapes and different mechanical properties for the reinforcing bars and the concrete.

Purpose. To determine the effects of the mechanical properties of high-strength reinforcement on columns, and to evaluate if the current limit on yield strength for compression reinforcement can be removed. In addition, this study is intended to evaluate whether traditional design assumptions for the development of axial-moment (P-M) interaction curves are appropriate with the use of high-strength reinforcement.

Details of Study. This study consists of analytical investigations to develop moment-curvature relationships for columns reinforced with high-strength reinforcement, and to evaluate the effects of different mechanical properties and stress-strain curve shapes. These studies can be used to complement the studies outlined in Section 4.4.1. In addition, this study should investigate the effects of creep and the transfer of stress from the concrete to the reinforcement. The following variables should be considered:

- Concrete compressive strength (varying from 4,000 psi to 12,000 psi)
- Confined and unconfined concrete
- Yield strength of reinforcement
- Reinforcement tensile-to-yield strength ratio
- Stress-strain curve shape
- Reinforcement ratio

- Axial load ratio
- Ratio of the confined core to overall cross-section dimensions

Recommendations regarding the relaxation of the current limit on yield strength for compression reinforcement should be provided.

Team. One researcher and one part-time graduate student.

Timeline. Approximately 3 months (assumed to occur in combination with the study outlined in Section 4.4.1).

4.4.4 Tension Regions of Shells and Folded Plates

Experimental and analytical studies to investigate potential reduction in stiffness and associated increases in tensile strain and cracking from the use of high-strength reinforcement in shells and folded plates.

Purpose. To determine the effects of potential stiffness reductions, increases in bar strains, and increases in cracking that might occur in the tension regions of shells and folded plates. Testing of reinforced concrete panels will be used to quantify the reduced tension stiffness, increase in bar strains, and increase in cracking, and finite element analyses will use the results from testing to evaluate the effects.

Details of Experimental Study. This study consists of experimental testing to explore the effect of tensile strength of concrete, shrinkage of concrete, bar size, and duration of loading. Tensile strength and shrinkage effects can be assessed by using different concrete mix designs (e.g., three). At least two different bar sizes should be used in the panels to determine whether a bar size effect exists. At least two panels should be placed in constant tension for a period of approximately one year to determine the effect of long-term loading. The number of tests should be established to strategically test for different parameters. For budgeting purposes, the number of test specimens is anticipated to be on the order of 10 panels.

Details of Analytical Study. This study consists of analytical investigations on shells and folded plates in which tension stiffness is varied based on the test results to determine the effects on the behavior. For budgeting purposes, approximately five models of various geometries should be analyzed.

Team. One researcher, one full-time graduate student (three years), one part-time graduate student (one year), and one part-time engineering practitioner for structural advice and consultation.

Timeline. Approximately 3 years.

4.4.5 Shear Strength of Beams and Slabs without Shear Reinforcement

Experimental studies to determine whether current design expressions for calculation of the concrete contribution to shear strength, V_c, are appropriate for flexural members without shear reinforcement, but incorporating the use of high-strength longitudinal reinforcement.

4.4.5.1 One-Way Shear in Beams without Shear Reinforcement

An experimental study on shear strength of beams without shear reinforcement.

Purpose. To evaluate current one-way shear strength equations for beams without shear reinforcement, but with high-strength longitudinal reinforcement. It is anticipated that testing will be targeted to verify that test results from specimens utilizing high-strength reinforcement fit within the band of currently available test results.

Details of Study. This study consists of experimental testing to evaluate the one-way shear strength of reinforced concrete beams incorporating high-strength longitudinal reinforcement. For this program, rectangular, simply supported beams subjected to a region of constant shear, would be appropriate. In addition, a limited number of continuous beam tests should be performed to evaluate the influence of the moment-to-shear (M/V) ratio, and to provide insight on behavior under conditions of high shear accompanied by high moment. The following variables should be considered:

- Reinforcement ratio (e.g., 0.25%, 0.5%, 0.75%, 1%)
- Yield strength of reinforcement (e.g., Grades 60, 100, and 120)
- Concrete compressive strength (e.g., 5,000 psi, 10,000 psi)
- Member depth (e.g., 12 inches, 24 inches, and 48 inches)
- Moment-to-shear (M/V) ratio

For budgeting purposes, the number of test specimens is anticipated to be on the order of 30.

Team. One researcher, one full-time graduate student (three years), one part-time graduate student (18 months), and one part-time laboratory technician to assist.

Timeline. Approximately 3 years.

4.4.5.2 Two-Way Shear in Slabs without Shear Reinforcement

An experimental study on shear strength of slabs without shear reinforcement.

Purpose. To evaluate current two-way shear strength equations for slabs without shear reinforcement, but with high-strength longitudinal reinforcement. It is anticipated that testing will be targeted to verify that test results from specimens utilizing high-strength reinforcement fit within the band of currently available test results.

Details of Study. This study consists of experimental testing to evaluate the two-way punching shear strength of reinforced concrete slabs incorporating high-strength longitudinal reinforcement. For this program, standard punching shear specimens with a concentrated load applied through a column stub, would be appropriate. Both concentric axial loading and combined flexure-axial loading should be considered to evaluate the influence of high shear accompanied by high moment. The following variables should be considered:

- Reinforcement ratio (e.g., 0.25%, 0.5%, 0.75%, 1%)
- Yield strength of reinforcement (e.g., Grades 60, 100, and 120)
- Concrete compressive strength (e.g., 5,000 psi, 10,000 psi)
- Member depth (e.g., 4 inches, 12 inches, and 24 inches)
- Moment-to-shear (M/V) ratio

For budgeting purposes, the number of test specimens is anticipated to be on the order of 20.

Team. One researcher, one full-time graduate student (three years), one part-time graduate student (18 months), and one part-time laboratory technician to assist.

Timeline. Approximately 3 years.

4.4.6 Shear Strength of Beams with Shear Reinforcement

An experimental study to determine if shear strength design expressions for calculation of V_n are appropriate for flexural members incorporating both longitudinal and transverse high-strength reinforcement, and to investigate the requirement for minimum area of shear reinforcement.

Purpose. To evaluate: (1) if yield strength is appropriate for use in the design equation for V_s, and if the combination of the concrete contribution,

V_c, and the shear reinforcement contribution, V_s, remains applicable; (2) crack widths and serviceability issues associated with high-strength shear reinforcement; and (3) the adequacy of the current requirement for minimum area of shear reinforcement.

Details of Study. This study consists of experimental testing to evaluate whether or not the yield strength of transverse reinforcement can be fully developed in resisting shear, the crack widths resulting from the use of shear reinforcement in excess of 60 ksi, and the need for minimum area of shear reinforcement. For this program, rectangular, simply supported beams subjected to a region of constant shear, would be appropriate. In addition, a limited number of continuous beam tests should be performed to evaluate the influence of the moment-to-shear (M/V) ratio, and to provide insight on behavior under conditions of high shear accompanied by high moment. This study can be used to complement the tests of beams without shear reinforcement outlined in Section 4.4.5.1. The following variables should be considered:

- Spacing of transverse reinforcement spacing
- Yield strength of transverse reinforcement (e.g., Grades 60, 100, and 120)
- Transverse reinforcement stress-strain curve shape
- Longitudinal reinforcement ratio (e.g., 0.5% and 1%)
- Yield strength of longitudinal reinforcement (e.g., Grades 60, 100, and 120)
- Concrete compressive strength (e.g., 5,000 psi, 10,000 psi)
- Member depth (e.g., 12 inches, 24 inches, and 48 inches)
- Shear span-to-depth (a/d) ratio (greater than 3)
- Moment-to-shear (M/V) ratio

For budgeting purposes, the number of test specimens is anticipated to be on the order of 40.

Team. One researcher, one full-time graduate student (three years), one part-time graduate student (18 months), and one part-time laboratory technician to assist.

Timeline. Approximately 3 years.

4.4.7 Shear-Friction

An experimental study to determine if the full yield strength of high-strength shear-friction reinforcement can be realized in producing normal forces across a shear transfer plane.

Purpose. To evaluate if the yield strength of high-strength shear-friction reinforcement is appropriate for use in the design of shear strength across an interface, or if a limit on the stress developed by the reinforcement is needed.

Details of Study. This study consists of experimental testing to evaluate the shear strength along an interface with high-strength shear-friction reinforcement. For this program, reinforcement placed perpendicular to the shear plane (consistent with the majority of cases), would be appropriate, and specimens should be suitably sized to allow for realistic bar spacing as well as full anchorage of the reinforcement. A variety of interfaces should be considered, including: (1) concrete placed monolithically; (2) concrete placed against hardened concrete with surface intentionally roughened; (3) concrete placed against smooth hardened concrete; and (4) concrete placed against structural steel. The following additional variables should be considered:

- Percentage of shear-friction reinforcement across the interface
- Size of shear-friction reinforcing bars
- Yield strength of shear-friction reinforcement (e.g., Grades 60, 100, and 120)
- Concrete compressive strength (e.g., 5,000 psi, 10,000 psi)

For budgeting purposes, the number of test specimens is anticipated to be on the order of 30.

Team. One researcher, one full-time graduate student (two years), and one part-time laboratory technician to assist.

Timeline. Approximately 3 years.

4.4.8 High-Cycle Elastic Fatigue of High-Strength Reinforcing Bars

An experimental study to determine the high-cycle fatigue resistance of high-strength reinforcement.

Purpose. To determine whether changes to the American Association of State Highway and Transportation Officials (AASHTO) *LRFD Bridge Design Specifications* (AASHTO, 2014) are required for high-cycle elastic fatigue resistance.

Details of Study. This study consists of experimental testing for high-cycle fatigue resistance of high-strength reinforcing bars using in-air axial tension specimens (Jhamb and MacGregor, 1974; Paulson and Hanson, 1991; Fei and Darwin, 1999; Zheng and Abel, 1999).

The effect of bar production method should be explored. Specimens should include bars produced by both quenching and tempering and micro-alloying, with bars obtained from multiple producers and in multiple sizes for each production method. The radius at the base of deformations, where the transverse lug meets the barrel of the bar, should be measured (Helgason et al., 1976). Representative bars should be examined to identify the presence and severity of defects at the base of the deformations. The range of bar sizes to be tested should consider both historical precedence and also the size of bars likely to be commonly specified for bridge flexural members with high-strength reinforcement subjected to elastic fatigue stresses in tension.

For budgeting purposes, the number of test specimens is anticipated to be on the order of 100 unspliced, high-strength reinforcing bars.

Team. One researcher and two full-time graduate students.

Timeline. Approximately 3 years.

4.5 Serviceability

Studies related to serviceability are summarized in Table 4-3.

Table 4-3 Studies Related to Serviceability

Reference Section	*Research or Engineering Study*	*Estimated Timeline*
4.5.1	Deflection of Flexural Members	3 years
4.5.2	Crack Control of Flexural Members	2 years

4.5.1 Deflection of Flexural Members

An analytical study on the deflection of flexural members reinforced with high-strength reinforcement, and an experimental study on the long-term deflection of flexural members.

Purpose. To investigate ACI 318 procedures for controlling deflections in flexural members. Specifically, potential adjustments to minimum thickness tables in ACI 318-14 for one-way and two-way flexural members (Tables 7.3.1.1, 8.3.1.1, 8.3.1.2, and 9.3.1.1) for beams and slabs reinforced with high-strength bars, and Equations 24.2.3.5a and 24.2.4.1.1 in ACI 318-14 for calculating deflections using an effective moment of inertia, I_e.

Details of Study. This study consists of analytical investigations for short-term deflections (Phase 1) and experimental testing for long-term deflections (Phase 2).

Phase 1: Short-Term Deflections

An analytical study should be conducted to predict the deflections of different flexural members including beams, one-way slabs, and two-way slabs. For one-way slabs, it is recommended that deflections be calculated using moment-curvature analysis considering the stress-strain relationship of the high-strength reinforcement. For two-way slabs, a finite element analysis, appropriately incorporating the moment-curvature behavior of the slab reinforced with high-strength reinforcement, is recommended. The two way-slab models should consider slabs with and without drop-caps. The following variables should be considered:

- Yield strength of reinforcement (e.g., Grades 60, 100, and 120)
- Concrete compressive strength (e.g., 4,000 psi, 8,000 psi, and 12,000 psi)
- Span length
- Member depth and width
- Reinforcement ratio
- Magnitude of service level loading

A study was conducted for the Charles Pankow Foundation, entitled *The Impact of High-Strength Reinforcing Steel on Current Design Practice* (Price et al., 2013), which computed deflections using finite element analysis on section properties that were varied. The results of this study can be used as a starting point for member sizes and reinforcement. In addition to parametric analyses, these analytical models should also be used to calculate deflections of beams that will be tested for one-way shear as described in Section 4.4.5.1, and slabs that will be tested for two-way shear as described in Section 4.4.5.2. Comparison between analytical and experimental results will enable validation of the accuracy of the models.

Minimum Depth Tables. Using results from the analytical study, and considering code-defined deflection limits, appropriate span-to-depth ratios should be developed and compared with the values presented in minimum depth tables in ACI 318-14. Recommended changes to Tables 7.3.1.1, 8.3.1.1, 8.3.1.2, and 9.3.1.1 in ACI 318 should be developed to address high-strength reinforcement.

Effective Moment of Inertia. Results from the analytical study should be compared with results from elastic theory using the effective moment of inertia method. Different approaches to computation of the effective moment of inertia should be considered, including the Branson equation (Equation 3-1), the Bischoff equation (Equation 3-2), and the cracked moment of inertia I_{cr}. For both the Branson and Bischoff equations, the use of a reduced cracking moment, M_{cr}, based on a reduction in the modulus of rupture, f_r, should be considered, as previous research has indicated that this parameter can be used to improve the estimation of deflections computed by the Branson equation. This approach may not be useful for the Bischoff equation, but should be considered. For two-way slabs, recommended finite element modeling techniques for the calculation of deflections should be provided.

Phase 2: Long-Term Deflections

An experimental study should be conducted to investigate long-term deflections dominated by creep. Although analytical models can be used to estimate creep effects, experimental testing provides the best information regarding this behavior. The design of the specimens should be influenced by the results of the analytical study, and beam specimens are recommended. Test specimens should be loaded to service levels, which will allow for comparison and verification of analytical models recommended in Phase 1. In addition, these same specimens should be subjected to sustained loads for one to two years, allowing for investigation of long-term deflections. The results of this testing should be compared with the results of the recommended design expression from Phase 1, amplified using the time-dependent factor from ACI 318. Appropriate changes to the time-dependent factor, if required, should be developed.

For budgeting purposes, the number of test specimens is anticipated to be on the order of six, consisting of three different longitudinal reinforcement ratios (e.g., 0.2%, 0.8 % and 1.5%), each tested at two different levels of sustained loading.

Team. One researcher, one full-time graduate student (three years), one part-time graduate student (18 months), one part-time laboratory technician to assist, and one part-time engineering practitioner for structural advice and consultation.

Timeline. Approximately 3 years.

4.5.2 Crack Control of Flexural Members

An analytical study of crack control of flexural members coupled with targeted experimental testing to confirm results.

Purpose. To investigate if the current design approach for control of cracking is appropriate at service level stresses higher than 40 ksi, which is the current default service stress in ACI 318.

Details of Analytical Study. This study consists of analytical investigations to evaluate crack widths that are developed at service level stresses greater than 40 ksi. It would be appropriate to consider crack widths developed up to the linear limit of the reinforcement to provide complete understanding of the range of behavior in the linear range. The analytical study should consider the physical model developed by Frosch (1999), which is the basis of the current ACI 318 design approach, and evaluate it for the following parameters: (1) service stress level; (2) concrete cover; and (3) reinforcement spacing.

Details of Experimental Study. This study consists of limited experimental testing with high-strength reinforcement to validate the accuracy of the analytical model. Specimens should consist of simply supported beams subjected to a constant moment region. As loading is increased, crack widths should be measured at multiple stress levels. The following variables should be considered:

- Reinforcement spacing
- Yield strength of reinforcement (e.g., Grades 60, 100, and 120)
- Member type (e.g., 16-inch deep beams and 6-inch slabs)

For budgeting purposes, the number of test specimens is anticipated to be on the order of 10.

Team. One researcher, one full-time graduate student (two years), and one part-time laboratory technician to assist.

Timeline. Approximately 2 years.

4.6 Reinforcing Limits

Studies related to reinforcing limits are summarized in Table 4-4.

Table 4-4 Studies Related to Reinforcing Limits

Reference Section	*Research or Engineering Study*	*Estimated Timeline*
4.6.1	Minimum Reinforcement Ratio for Beams	2 months
4.6.2	Minimum Reinforcement Ratios for Slabs and Footings	2 months

4.6.1 Minimum Reinforcement Ratio for Beams

An analytical study on minimum flexural reinforcement in beams.

Purpose. To determine minimum flexural reinforcement required to provide an acceptable ratio of cracked section flexural strength to uncracked section strength in beams with high-strength reinforcement.

Provisions are currently being reworked by ACI Committee 318. If a direct solution (such as the current approach in Section 9.6.1.2 of ACI 318-14) is used, this study is needed. If an approach similar to current requirements for prestressed members (based on a multiple of the uncracked flexural capacity) is used, this study can be omitted.

Details of Study. This study consists of flexural section analyses to identify $A_{s,min}$, corresponding to $1.2(M_{n,cracked}/M_{n,uncracked})$. Trial designs should consider rectangular and T-sections along with the following variables: (1) ratio of beam depth-to-width; (2) ratio of reinforcement depth to beam depth; (3) concrete compressive strength; and (4) yield strength of flexural reinforcement (e.g., from 80 ksi to 120 ksi).

Team. One lead engineering practitioner and one part-time staff engineer to assist in trial designs and flexural section analyses.

Timeline. Approximately 2 months.

4.6.2 Minimum Reinforcement Ratios for Slabs and Footings

An analytical study on minimum flexural reinforcement in slabs and footings.

Purpose. To determine minimum flexural reinforcement required to provide an acceptable ratio of cracked section flexural strength to uncracked section strength in slabs and footings with high-strength reinforcement.

Current ACI 318 provisions place no limit on f_y in the calculation of minimum reinforcement for slabs and footings. If ACI Committee 318 implements a strength-based lower-bound on flexural reinforcement in slabs and footings, this study is needed.

Details of Study. This study consists of flexural section analyses to identify $A_{s,min}$, corresponding to $1.2(M_{n,cracked}/M_{n,uncracked})$. Trial designs should consider the following variables: (1) slab (or footing) depth; (2) concrete compressive strength; and (3) yield strength of flexural reinforcement (e.g., from 80 ksi to 120 ksi).

Team. One lead engineering practitioner and one part-time staff engineer to assist in trial designs and flexural section analyses.

Timeline. Approximately 2 months.

4.7 Detailing of Members

Studies related to detailing of members are summarized in Table 4-5.

Table 4-5 Studies Related to Detailing of Members

Reference Section	*Research or Engineering Study*	*Estimated Timeline*
4.7.1	Development and Splice Lengths	2.5 years
4.7.2	Hooked Bar Development Length	3 years
4.7.3	Headed Bar Development Length	2 years

4.7.1 Development and Splice Lengths

An experimental study on development and splice lengths for high strength reinforcement considering concrete compressive strength, bar spacing, and epoxy coating.

Purpose. To fill gaps in available test data on development and splice lengths of high-strength reinforcing bars in concrete with compressive strengths greater than 10,000 psi, bars with clear spacing less than two bar diameters, and bars with epoxy coating.

Details of Study. This study consists of splice tests at bar stresses of 80 ksi to 140 ksi. Three series of tests should be performed to determine the effects of concrete compressive strength (e.g., ranging from 8,000 psi to 15,000 psi), the effects of bar spacing (e.g., as low as one bar diameter), and the effects of epoxy coating. Tests should involve splices both with and without confining transverse reinforcement. The study should emphasize the effect of closely spaced bars to demonstrate the reduction in developed bar stresses.

Team. One researcher, two full-time graduate students, and one part-time laboratory technician to assist.

Timeline. Approximately 2.5 years.

4.7.2 Hooked Bar Development Length

An experimental study to supplement data from tests on the development length of high-strength hooked bars currently in progress at the University of Kansas. This study is intended to address the use of staggered hooks, which is not within the scope of the University of Kansas tests.

Purpose. To investigate the performance of conventional and high-strength hooked reinforcement when the hooks are closely spaced, staggered, or both, as permitted by ACI 318, and to determine the effect of larger bend diameters on anchorage strength. There is a possibility that bend diameters for standard hooks might be increased for fabrication purposes, which is why testing of hooked bars with larger bend diameters is included in this study. If standard hook diameters are not increased, the scope of this testing program can be reduced.

Details of Study. This study consists of tests on simulated beam-column joints involving the placement of hooks. The following variables should be considered: bar stress (e.g., between 80 ksi and 140 ksi); bar size (e.g., No. 5 to No. 11); and concrete compressive strength (e.g., ranging from 5,000 psi to 15,000 psi). The anchorage strength of hooks with larger diameter bends should be compared to that of conventional standard hooks. Bar spacing as low as one bar diameter, and staggered hooks in beam column joints, both with and without confining transverse reinforcement, should be tested. Limited tests should also be performed on deep beams with staggered hooks to investigate anchorage at nodes in strut-and-tie models.

Team. One researcher, two full-time graduate students, and one part-time laboratory technician to assist.

Timeline. Approximately 3 years (two years if hooks with larger bend diameters are not tested).

4.7.3 Headed Bar Development Length

An experimental study to supplement data from tests on the development length of high-strength headed bars currently in progress at the University of Kansas. This study is intended to address the use of closely spaced, staggered headed bars, which is not within the scope of the University of Kansas tests.

Purpose. To investigate the behavior of closely spaced, staggered headed bars, such as those used in beam-column joints and transfer girders, which require a high concentration of reinforcement that must be anchored within a short distance.

Details of Study. This study consists of tests on simulated beam-column joints and deep beams with closely spaced, staggered headed bars. The following variables should be considered: bar stress (e.g., between 80 ksi and 140 ksi); bar size (e.g., No. 5 to No. 11); concrete compressive strength (e.g., ranging from 5,000 psi to 15,000 psi); and net bearing area of the head (e.g., 4 to 14 times the area of the bar). Specimens both with and without confining transverse reinforcement should be tested.

Team. One researcher, two full-time graduate students, and one part-time laboratory technician to assist.

Timeline. Approximately 2 years.

4.8 General Considerations for Analysis

Studies related to general considerations for analysis are summarized in Table 4-6.

Table 4-6 Studies Related to General Considerations for Analysis

Reference Section	*Research or Engineering Study*	*Estimated Timeline*
4.8.1	Flexural Stiffness	2 years
4.8.2	Effective Stiffness for Column Slenderness	2 years
4.8.3	Moment Redistribution	2 years

4.8.1 Flexural Stiffness

An analytical study on flexural stiffness, *EI*, for analysis of a structure at the ultimate limit state for computation of design forces, and at the serviceability limit state for estimation of deflections and lateral drifts.

Purpose. To develop guidance on the selection of flexural stiffness, *EI*, for use in structural analysis.

Details of Study. This study consists of analytical investigations to evaluate the flexural stiffness, *EI*, required to reasonably estimate: (1) design forces at the ultimate limit state; and (2) deflections and drifts at the service limit state. As noted in the commentary of ACI 318, "*EI* values should not be based totally on the moment-curvature relationship for the most highly loaded section along the length of each member. Instead, they should correspond to the moment-end rotation relationship for a complete member." Structural analysis at the ultimate limit state should consider the extent of cracking at a section and along the length of a member so that reasonable relative stiffness values for columns, walls, and beams can be realized. Similarly, the level of cracking anticipated at service levels should be considered to develop guidance at this limit state. Overall, a single value of *EI* for each member type is needed to allow for simplified analysis of an entire structural system. This study should evaluate the differences in stiffness required for Grade 100 and Grade 120 reinforcement, as compared to Grade 60 reinforcement, and relevant test results from experimental studies outlined in this *Roadmap* should be used to calibrate the analytical work.

Team. One researcher, one full-time graduate student (two years), and one part-time engineering practitioner for structural advice, consultation, and design assistance.

Timeline. Approximately 2 years.

4.8.2 Effective Stiffness for Column Slenderness

An analytical study and confirming experimental tests on values of effective stiffness, *EI*, used to estimate column second-order effects.

Purpose. To determine if effective stiffness values used to design for column slenderness effects require adjustment, and if so, how to adjust them for members reinforced with high-strength reinforcement.

Details of Study. This study consists of analytical investigations to evaluate the effective stiffness, *EI*, required to properly estimate second-order effects in slender columns. The study should use moment-curvature analyses to characterize the column in combination with second-order analysis of slender-columns. Previous research has found that a critical column configuration consists of a small (i.e., 12 inch by 12 inch) column section with a reinforcement ratio of 1%. Primary variables should include the yield strength of the reinforcement, slenderness ratio of the column, boundary conditions, and the concrete compressive strength. A limited experimental study, on the order of five slender column test specimens, is recommended to provide confirmation of the analytical results.

Team. One researcher, one full-time graduate student (two years), and one part-time engineering practitioner for structural advice, consultation, and design assistance.

Timeline. Approximately 2 years.

4.8.3 Moment Redistribution

An analytical study on the ability of structures reinforced with high-strength reinforcement to redistribute moments.

Purpose. To evaluate the capability of structures with high-strength reinforcement to redistribute moments, and to determine if changes to current permissible moment redistribution percentages are needed.

Details of Study. This study consists of analytical investigations to determine the amount of redistribution that can be achieved when high-strength longitudinal reinforcement is provided. This study will benefit from the results of analyses and testing outlined in Section 4.4.2, as the plastic capacity of beams reinforced with different reinforcement materials will be developed. The following variables should be considered:

- Longitudinal reinforcement ratio (e.g., 0.5% to 2.0%)
- Yield strength of reinforcement (e.g., Grades 60, 80, 100, and 120)
- Structural geometry, including beam aspect ratio, span-to-depth ratio, and effective slab width (e.g., T-beams)
- Plastic hinge length

Team. One researcher and one full-time graduate student.

Timeline. Approximately 2 years.

4.9 Earthquake-Resistant Structures

Studies related to earthquake-resistant structures are summarized in Table 4-7.

4.9.1 Moment-Curvature and Rotational Capacity

An analytical study in which moment-curvature relationships are developed for beam, column, and wall elements used in seismic force-resisting systems.

Purpose. To determine reinforcement strain levels and rotational capacities for elements used in seismic-force-resisting systems. Results are intended to help define requirements for elongation and tensile-to-yield strength ratio for high-strength reinforcement in seismic applications.

Details of Study. This study consists of analytical investigations to extend moment-curvature and rotational capacity studies on beam, column, and wall elements presented in NIST GCR 14-917-30, *Use of High-Strength Reinforcement in Earthquake-Resistant Concrete Structures* (NIST, 2014).

These studies are intended to investigate possible variation in how plasticity spreads for different reinforcement stress-strain curve shapes and controlling parameters. The shape of the stress-strain curves used in this study should include low tensile-to-yield strength ratios and different values for usable

Table 4-7 Studies Related to Earthquake-Resistant Structures

Reference Section	*Research or Engineering Study*	*Estimated Timeline*
4.9.1	Moment-Curvature and Rotational Capacity	18 months
4.9.2	Factor for Estimating Expected Flexural Strength	18 months
4.9.3	Cyclically Loaded Beams and Columns – Initial Tests and Analytical Studies	2 years
4.9.4.1	Cyclically Loaded Beams	3 years
4.9.4.2	Cyclically Loaded Columns	3 years
4.9.4.3	Cyclically Loaded Interior Joints	3 years
4.9.4.4	Cyclically Loaded Exterior Joints	3 years
4.9.4.5	Two-Way Shear in Slab-Column Intermediate Moment Frames	3 years
4.9.5	Performance of Moment Frames Systems	2 years
4.9.6	Multi-Bay, Multi-Story Frames	3 years
4.9.7	Ordinary Flexure-Critical Walls	3 years
4.9.8	Special and Ordinary Shear-Critical Walls	5 years
4.9.9	Special Flexure-Critical Walls – Initial Tests	2 years
4.9.10	Special Flexure-Critical Walls	4 years
4.9.11	Performance of Flexure-Critical Wall Systems	2 years

strain. Plastic hinge lengths should be varied to account for the effects of tensile-to-yield strength ratio on yield penetration in the members, and the strain gradient within the plastic hinge length should also be considered.

The following variations in the shape of the stress-strain curves for Grade 80, Grade 100, and Grade 120 reinforcement are recommended:

- Bilinear curves with tensile-to-yield strength ratios of 1.10, 1.15, and 1.20, and uniform elongation of 5%.
- Curves with a yield plateau of 0.5% length, tensile-to-yield strength ratios of 1.10, 1.15, 1.20, 1.25, and 1.35, and uniform elongation of 5%, 7%, and 8%.
- Rounded curves with yield strength based on 2% offset, tensile-to-yield strength ratios of 1.15, 1.20, 1.25, 1.35, and 1.50, and uniform elongation of 5%, 7%, and 8%.

As a minimum, this study should consider the results of tests described in Section 4.9.3 to determine plastic hinge length, bar strain, and spread of plasticity within the plastic hinge region. For consistency with typical U.S. practice, beams and slabs should have unequal top and bottom areas of reinforcement.

Team. One researcher and one part-time graduate student.

Timeline. Approximately 18 months.

4.9.2 Factor for Estimating Expected Flexural Strength

An analytical study to perform moment-curvature analyses and numerical simulations on typical moment frame beam sections to evaluate nominal and plastic (expected) flexural strengths.

Purpose. To establish an appropriate factor, currently 1.25 for Grade 60 reinforcement, to estimate plastic moment capacity from the nominal flexural strength of members reinforced with high-strength reinforcement. This ratio is used to compute capacity-based shear force demands on beams and beam-column joints, and flexural and shear force demands on supporting columns, avoiding the need for moment-curvature calculations for which there is no guidance in ACI 318.

Details of Study. A two-phase analytical study is recommended to establish the ratio between plastic flexural capacity and nominal flexural strength of typical beams.

Phase 1. In this first phase, engineering practitioners should be used to assemble a database of moment frame beam details, organized by reinforcement grade, concrete strength, beam span-to-depth ratio, effective slab width and depth, and transverse reinforcement details, based on a survey of past projects. Beam sections in the database should then be redesigned using two grades of high-strength reinforcement, with the goal of achieving the same nominal strength, while other details remain unchanged. Moment-curvature analyses for positive and negative bending should be performed on all beam sections in the database to establish the ratio of plastic (maximum) flexural strength to nominal strength. The effects of confinement and rebar strain hardening should be considered explicitly, and nominal strength should be based on the rectangular cross section only.

Phase 2. In this second phase, rotation demands and maximum curvatures on beams in special moment frames should be estimated for the design basis and maximum considered earthquake hazard levels. Resulting estimates of curvature, which will vary with span-to-depth ratio, should be used as the basis for calculating maximum considered strength (less than plastic flexural strength) and the multiplier on nominal strength.

Team. Two engineering practitioners, one researcher, and one part-time graduate student to assist.

Timeline. Approximately 2 years, assuming 18 months for Phase 1, and an additional 6 months for Phase 2.

4.9.3 Cyclically Loaded Beams and Columns – Initial Tests and Analytical Studies

An analytical and experimental study including initial cyclic load tests on beam and column specimens. Results are intended to help define mechanical properties for high-strength reinforcement in seismic applications.

Purpose. To determine rotational capacity of beams and columns reinforced with high-strength reinforcement with tensile-to-yield ratios and elongations that are less than the requirements for ASTM A706 Grade 80 reinforcing bars. Analytical results will be used to represent demands at incipient collapse, which will then be used to judge the acceptability of test results representing the capacity of members with high-strength reinforcement.

Details of Analytical Study. This study consists of nonlinear response history analyses to establish rotational demands in beams and columns in a moment frame system just prior to lateral instability (i.e., sidesway collapse).

Details of Experimental Study. This study consists of cyclic testing of specimens loaded to failure. Strains in the reinforcing bars and length of yielding should be monitored to determine equivalent plastic hinge lengths, and bar slip into the joint should also be monitored. Response data should be recorded in sufficient detail to be used in developing nonlinear component models for collapse analyses, and to improve moment-curvature analyses described in Section 4.9.1.

Tested mechanical properties of the reinforcing bars and the concrete must be well documented. For example, the extensometer should not be removed from the bars until after peak strength is reached (i.e., beyond the uniform elongation). Tests should focus on determining the minimum tensile-to-yield strength ratio of high-strength reinforcement required to obtain sufficient spread of yielding, the uniform elongation required, and the low-cycle fatigue resistance necessary for satisfactory seismic performance.

For budgeting purposes, the number of test specimens is anticipated to be on the order of four beams and four columns. Each beam or column set should consist of two pairs of tests, one with Grade 100 reinforcement and the second pair with either Grade 100 or Grade 120 reinforcement.

Team. Two teams (separate research facilities assumed), each consisting of one researcher, one full-time graduate student, and one part-time laboratory technician to assist.

Timeline. Approximately 2 years, assuming 1 year to conduct testing, and 1 year to process results.

4.9.4 Cyclically Loaded Beams, Columns, and Joints

Experimental studies on cyclically loaded beams, columns, interior joints, and exterior joints.

4.9.4.1 Cyclically Loaded Beams

An experimental study to perform cyclic load tests on beams reinforced with high-strength reinforcement.

Purpose. To establish strength and rotational capacities of beams in special and intermediate moment frames. Specimens should be tested to failure so that nonlinear force-displacement response can be established and used in the development of nonlinear component models for response history analysis.

Details of Study. This study consists of cyclic load tests on beams in special and intermediate moment frames. Specimens should be representative of moment frame beams used in practice, including some large specimens to explore the effect of beam depth, an appropriate range of shear span-to-depth ratios, and appropriate range of moment-to-shear ratios. The following variables should be considered:

- Concrete compressive strength (varying from 5,000 psi to 10,000 psi)
- Yield strength of reinforcement (varying from 80 ksi to about 135 ksi, to account for overstrength and the effect of the shape of the stress-strain curve)
- Tensile-to-yield strength ratio of reinforcement (e.g., minimum specified to maximum actual)
- Reinforcement ratio
- Transverse reinforcement configuration and spacing

The specimens should be instrumented so that bar strains are recorded over the duration of the tests. This information will be used, in part, to establish the required usable strain in the bars.

For budgeting purposes, the number of test specimens is anticipated to be on the order of 16 beams. Approximately 20% to 30% of the tests should be beams of intermediate moment frames.

Team. Two teams (separate research facilities assumed), each consisting of one researcher, two full-time graduate students, and one part-time laboratory technician to assist.

Timeline. Approximately 3 years.

4.9.4.2 Cyclically Loaded Columns

An experimental study to perform cyclic load tests on columns reinforced with high-strength reinforcement.

Purpose. To establish strength and rotational capacities of columns in special and intermediate moment frames. Specimens should be tested to failure so that nonlinear force-displacement response can be established and used in the development of nonlinear component models for response history analysis.

Details of Study. This study consists of cyclic load tests on columns in special and intermediate moment frames. Specimens should be representative of moment frame columns used in practice, considering possible changes in required beam size and joint depth for high-strength reinforcement, and including an appropriate range of moment-to-shear ratios. The following variables should be considered:

- Concrete compressive strength (varying from 5,000 psi to 12,000 psi)
- Yield strength of reinforcement (varying from 80 ksi to about 135 ksi, to account for overstrength and the effect of the shape of the stress-strain curve)
- Tensile-to-yield strength ratio of reinforcement (e.g., minimum specified to maximum actual)
- Post-yield stiffness of reinforcement
- Transverse reinforcement configuration and spacing

For budgeting purposes, the number of test specimens is anticipated to be on the order of 16 columns. Approximately 20% to 30% of the tests should be columns of intermediate moment frames.

Team. Two teams (separate research facilities assumed), each consisting of one researcher, two full-time graduate students, one part-time laboratory technician to assist, and one engineering practitioner to provide structural advice and consultation.

Timeline. Approximately 3 years.

4.9.4.3 Cyclically Loaded Interior Joints

An experimental study to perform cyclic load tests on beam-column subassemblages of interior joints in special and intermediate moment frames.

Purpose. To investigate the required joint depth and the performance of interior beam-column joints reinforced with high-strength reinforcement, and compare results to joints with Grade 60 reinforcement.

Details of Study. This study consists of experimental testing of interior joint subassemblies. Specimens should be cross-shaped, with beam and column sizes consistent with the beam sizes tested in Section 4.9.4.1 and column sizes tested in Section 4.9.4.2. Joint depths should be selected to investigate recommendations in NIST GCR 14-917-30, and the recommendations established by Japan's New RC Project (Aoyama, 2001), to explore whether these recommendations result in joints that are consistent with the performance of joints with Grade 60 reinforcement.

Specimens should include a range of concrete compressive strengths (e.g., 5,000 psi, 8,000 psi, and 12,000 psi) and a range of axial load ratios, $P/f'_c A_g$, (e.g., 0.2, 0.4, and a ratio representing the maximum axial load at large drifts from FEMA P-695 analyses), as these two factors are likely to affect bar slip, and degradation due to bar slip, through the joint. The yield strength of reinforcement should be varied (e.g., Grades 80, 100, and 120), and joint depths should vary with the grade of bar. The following joint depths can be considered as a starting point for future consideration: joint depth of $26d_b$ for Grade 80; joint depths of $32d_b$ to $36d_b$ for Grade 100; and joint depths of $32d_b$ to $40d_b$ for Grade 120.

Tests should primarily consist of special moment frame joints, referred to as Type 2 joints in ACI 352R-02, *Recommendations for Design of Beam-Column Connections in Monolithic Reinforced Concrete Structures* (ACI, 2002b). A limited number of intermediate moment frame joints, referred to as Type 1 joints in ACI 352R-02, should also be tested. The number of tests should be sufficient to establish a joint depth criterion applicable to multiple grades of reinforcement, concrete strengths, and column axial load. For budgeting purposes, the number of test specimens is anticipated to be on the order of 12.

Team. Two teams (separate research facilities assumed): one team consisting of one researcher, two full-time graduate students, and one part-time laboratory technician to assist; and the other team consisting of one researcher, one full-time graduate student, and one part-time laboratory technician to assist.

Timeline. Approximately 3 years, assuming 2 years to conduct testing, and 1 year to process results.

4.9.4.4 Cyclically Loaded Exterior Joints

An experimental study to perform cyclic load tests on beam-column subassemblages of exterior joints in special and intermediate moment frames.

Purpose. To investigate joint depths based on development length for hooked and headed bars terminating at the joint, the amount of joint reinforcement needed to prevent joint splitting, and the applicability of the 1.25 factor on longitudinal bar yield strength used to compute the joint shear for high-strength reinforcement.

Details of Study. This study consists of experimental testing of exterior joint subassemblies. Specimens should be T-shaped with beam, column, and joint sizes representative of frames used in practice. Required development lengths should be computed based on required development lengths for high-strength reinforcement. Bars should have tensile-to-yield strength ratios similar to the maximum expected for high-strength reinforcing bars to increase the likelihood of bar slip.

Specimens should be constructed with longitudinal beam bars that terminate both with 90 degree hooks and with heads. Specimens using heads should include: (1) bars terminating with both Class HA and Class HB bar-to-head connections to investigate the need for stronger bar-to-head connections; and (2) heads with a net bearing areas between $4A_b$ and $9A_b$. The number and spacing of beam and column bars should be consistent with reinforcing details used in practice (e.g., 3-inch center-to-center spacing). Joints depths should be determined considering the following variables: (1) at least two different bar sizes (e.g., No. 6 and No. 9); (2) yield strength of reinforcement (e.g., Grades 80, 100, and 120); and (3) concrete compressive strength (e.g., 6,000 psi and 12,000 psi).

Selected specimens should include bars with the largest actual tensile-to-yield strength ratios expected to be produced for use in seismic applications. These specimens are intended to study the applicability of the 1.25 factor on yield strength used in computing joint shear. These specimens can also be used to determine if splitting of the joint is a problem with high-strength longitudinal bars, and if additional transverse reinforcement is required.

Tests should primarily consist of special moment frame joints, but a limited number of intermediate moment frame joints should also be tested. The test program should include enough tests to confirm that adequate performance of exterior joints will be achieved. For budgeting purposes, the number of test specimens is anticipated to be on the order of 12.

Team. Two teams (separate research facilities assumed), one team consisting of one researcher, two full-time graduate students, and one part-time laboratory technician to assist, and the other team consisting of one researcher, one full-time graduate student, and one part-time laboratory technician to assist.

Timeline. Approximately 3 years, assuming 2 years to conduct testing, and 1 year to process results.

4.9.4.5 Two-Way Shear in Slab-Column Intermediate Moment Frames

An experimental study to perform cyclic load tests on slab-column intermediate moment frame joints.

Purpose. To investigate the two-way shear strength of slab-column joints under cyclic loading, specifically the $0.4\phi V_c$ limit on two-way shear from gravity loading, which is used to protect against shear failure at story drifts up to 1.5%.

Details of Study. This study consists of experimental testing on slab-column joint specimens similar to the four tested by Pan and Moehle (1989), which were used, along with other tests, to establish the $0.4\phi V_c$ limit. Longitudinal reinforcement should be the maximum grade appropriate for two-way slabs (e.g., Grade 120) as this will minimize the reinforcement ratio and maximize strain in the bars. Slab shrinkage in the specimens should be restrained to replicate field conditions. The moment-shear ratio at the joint should be consistent with story drift demands expected in structures under seismic loading.

Team. One researcher, two full-time graduate students, and one part-time laboratory technician to assist.

Timeline. Approximately 3 years.

4.9.5 Performance of Moment Frame Systems

An analytical study on the collapse resistance of moment frame systems constructed using high-strength reinforcement.

Purpose. To assess the collapse resistance of moment frame systems reinforced with high-strength reinforcement. This study is needed to demonstrate that moment frame systems incorporating the use of high-strength reinforcement are capable of providing a level of safety that is consistent with the seismic performance intended in ASCE/SEI 7-10, and consistent with the performance of seismic-force-resisting systems that have traditionally been designed and constructed using Grade 60 reinforcement. It is intended to address questions regarding overall system behavior that are expected from the National Earthquake Hazard Reduction Program (NEHRP) Provisions Update Committee (PUC), and the ASCE/SEI 7 Seismic Code Committee, when considering reference to future editions of ACI 318 specifically incorporating the use of high-strength reinforcement.

Details of Study. This study consists of nonlinear response history analyses on a series of moment frame buildings using the methodology described in FEMA P-695, *Quantification of Building Seismic Performance Factors* (FEMA, 2009). Model parameters used in this study should be calibrated based on results from component and system tests outlined in Section 4.9.4.

FEMA P-695 analyses should be performed on prototype buildings with special and intermediate moment frame systems designed using high-strength reinforcement. Both beam-column and slab-column frames should be considered in the case of intermediate moment frame systems. Models that were developed in prior studies considering Grade 60 reinforcement (FEMA, 2009; NIST, 2010; and NIST, 2012), could be adapted for use with high-strength reinforcement. Building archetypes from prior studies, including low-rise, medium-rise and high-rise frames, should be redesigned with Grade 80, Grade 100, and Grade 120 reinforcement, and sections should be optimized consistent with standard design practice. Buildings should be sited in regions of moderate and high seismicity.

Results will be used to compare the collapse resistance of moment frame buildings with high-strength reinforcement to the criteria defined in FEMA P-695. A specific question to be addressed is whether or not the strong column-weak beam provisions require adjustment with the use of high-strength reinforcement.

If acceptable collapse resistance is not achieved, the study should identify design measures, material properties, or detailing requirements that would serve to improve collapse behavior. Such measures could be used as the basis for the development of design and detailing rules for special and intermediate moment frame systems with high-strength reinforcement that would: (1) provide acceptable resistance to collapse; and (2) be suitable for inclusion in ACI 318.

Team. One researcher, one full-time graduate student, and two part-time engineering practitioners to provide advice, consultation, and design assistance.

Timeline. Approximately 2 years.

4.9.6 Multi-Bay, Multi-Story Frames

An experimental study to perform cyclic load tests on multi-bay, multi-story special moment frames reinforced with Grade 100 reinforcement.

Purpose. To confirm the expected global performance of provisions for special moment frames. The effectiveness of specific provisions for strong column-weak beam behavior, probable moment strength, shear demand, joint depth, and joint shear strength should be monitored.

Details of Study. This study consists of experimental testing of large-scale, multi-bay, multi-story special moment frames. Frame specimens should be representative of frame configurations used in design practice. They should be two-dimensional, two-bay, two- or three-story special moment frames, with partial-width slabs to capture slab contributions to strength. Reinforcement should be Grade 100, and concrete compressive strengths should be varied (e.g., 6,000 psi and 12,000 psi). The specimens should be designed without added conservatism. For budgeting purposes, the number of test specimens is anticipated to be on the order of two frames.

Team. One researcher, two full-time graduate students, one part-time laboratory technician, and one part-time engineering practitioner to provide advice, consultation, and design assistance.

Timeline. Approximately 3 years.

4.9.7 Ordinary Flexure-Critical Walls

An experimental study to perform cyclic load tests on ordinary flexure-critical shear walls to evaluate performance of high-strength reinforcement and applicability to current code provisions for shear, *P-M* interaction, and possibly confinement.

Purpose. To evaluate the performance of high-strength reinforcement in flexure-critical shear walls subjected to minor and moderate ductility demands. Current ACI 318 provisions limit the yield strength of reinforcement to Grade 60 in shear, and Grade 80 in combined axial and flexure. Current provisions could be validated for use with high-strength reinforcement, or revised to accommodate differing behavior.

Details of Study. This study consists of experimental testing of ordinary flexure-critical walls for shear, *P-M* interaction, and confinement.

Shear. Perform cyclic lateral load tests on ordinary shear walls with aspect ratios of 3:1 to 4:1. The primary variable in this study is the quantity of horizontal reinforcement. Vertical reinforcement should be sized to ensure that shear is the primary mechanism. Reinforcement yield strengths should include Grade 100 and Grade 120. The goal is to explore the validity of V_s calculations in Sections 11.9.5, 11.9.6, and 11.9.9 of ACI 318-11 for high-strength reinforcement.

P-M Interaction. Perform cyclic lateral load tests on ordinary shear walls with aspect ratios in the range of 3:1 to 4:1. The primary variable in this study is the vertical reinforcement ratio (e.g., ranging from 0.0025 to 0.03). Horizontal reinforcement should be sized to ensure that *P-M* interaction is the primary mechanism. Reinforcement yield strengths should include Grade 100 and Grade 120. Axial load ratio, $P/(A_g f'_c)$, could also be varied (e.g., ranging from 0.05 to 0.30). The goal is to confirm the validity of standard *P-M* interaction equations for high-strength reinforcement. Additionally, Grade 60 specimens should be used as benchmarks for evaluating *P-M* ductility and hysteretic energy absorption, to which high-strength reinforcement specimens will be compared. The resulting ductility should also be compared to that intended for ordinary walls in ACI 318 and ASCE/SEI 7.

Confinement. Perform tests on walls with vertical bars stronger than the current limit of 80 ksi, which could be potentially more critical for bar buckling. These tests could also be conducted using column specimens. The goal is to validate current provisions, which includes a trigger requiring ties to prevent bar buckling in portions of walls with vertical reinforcement ratios greater than 0.01.

For budgeting purposes, the total number of test specimens in this study is anticipated to be on the order of 10, distributed between shear, *P-M* interaction, and confinement studies to optimize coverage of the recommended parameters.

Team. Two teams (separate research facilities assumed), each consisting of one researcher, one full-time graduate student, one part-time laboratory technician to assist, and one engineering practitioner to provide structural advice, consultation, and design assistance.

Timeline. Approximately 3 years.

4.9.8 Special and Ordinary Shear-Critical Walls

An experimental study on low aspect ratio, shear-critical walls, with and without boundary columns and flanges.

Purpose. To establish if the prescriptive details in ACI 318 for ordinary and special shear-critical walls are applicable if high-strength reinforcement is used. The goal of this testing program is to investigate changes in seismic performance, measured in terms of hysteretic behavior, if Grade 60 reinforcement is replaced by high-strength reinforcement. Determining acceptable hysteretic behavior for a shear-critical wall is not part of the scope of this study.

Details of Study. This study consists of a two-phase experimental program of reverse-cyclic, pseudo-static tests on walls with aspect ratios of 0.33 and 0.67, concrete compressive strengths of 3,500 psi and 6,000 psi, vertical and web reinforcement ratios of 0.25%, and Grade 60 (benchmark) and high-strength reinforcement grades. Wall specimens with Grade 60 and high-strength reinforcement should have the same nominal shear strength, which should be accomplished by varying the area and spacing of reinforcing bars. Phase 1 tests will examine walls with rectangular cross-sections, and Phase 2 tests will examine walls with boundary columns and flanges, if necessary.

Testing should be preceded by a literature review and analysis of prior tests of large-scale walls (e.g., testing conducted under the NEESR program). Work should include numerical simulation studies using OpenSees (OpenSees, 2011), LS-DYNA (LSTC, 2013), or DIANA (TNO, 2013).

For budgeting purposes, the number of test specimens in Phase 1 is anticipated to be on the order of 16 ordinary and special walls. If appreciable differences are observed in the behavior of nominally identical (i.e., in shear strength) walls with Grade 60 and higher strength reinforcement, approximately six to eight additional specimens with boundary columns and flanges should be tested in Phase 2.

Team (Phase 1). Two teams (separate research facilities assumed), one team consisting of one researcher and three full-time graduate students (three years) to conduct testing and numerical studies, and the other team consisting of one researcher and two full-time graduate students (three years) to conduct testing, along with one part-time engineering practitioner to provide structural advice, consultation, and design assistance on each team.

Team (Phase 2). One team (at a third research facility), consisting of one researcher, two full-time graduate students (three years) to conduct testing and numerical studies, and one part-time engineering practitioner to provide structural advice, consultation, and design assistance.

Timeline. Approximately 5 years, assuming 3 years to complete Phase 1 tests and 3 years to complete Phase 2 tests, in a staggered schedule.

4.9.9 Special Flexure-Critical Walls – Initial Tests

An initial experimental study on special, flexure-critical shear walls, including large-scale flanged walls subjected to inelastic displacement reversals.

Purpose. To determine the required minimum uniform elongation and tensile-to-yield strength ratio for the safe use of high-strength reinforcement in seismic-force-resisting systems. The strength, stiffness, deformation capacity, and hysteretic response of specimens should be investigated.

Details of Study. This study consists of cyclic testing of flexure-critical concrete shear walls constructed with high-strength longitudinal and transverse reinforcement. Primary variables include the specified yield strength of the wall reinforcement (e.g., Grade 60 and Grade 100), and the tensile-to-yield strength ratio of the wall reinforcement (varying from 1.1 to 1.5). Design of the specimens should comply with ACI 318-14 requirements for special structural walls, as well as the additional detailing requirements identified in NIST GCR 14-917-30. All specimens should have aspect ratios (height-to-length) not less than 3:1, and the applied axial load (if any) should be held constant throughout the test.

This study should address: (1) strength, especially the influence of the shape of the stress-strain curves of the wall reinforcement on flexural strength and on the variation of concrete and steel contributions to shear strength as a function of deformation demands; (2) stiffness, with emphasis on characterizing the reduction of unloading and reloading lateral stiffness with increased deformation; (3) deformation capacity, including the effects of reinforcement strains on strength reduction; and (4) hysteretic response, identifying key parameters that influence the measured cyclic response and the variation in energy dissipation in successive cycles. For budgeting purposes, the number of test specimens in this initial test program is anticipated to be on the order of four walls.

Team. One researcher, two full-time graduate students, and one part-time laboratory technician to assist.

Timeline. Approximately 2 years, assuming 1 year to conduct testing, and 1 year to process results.

4.9.10 Special Flexure-Critical Walls

An experimental study on special, flexure-critical shear walls reinforced with high-strength reinforcement that meet newly developed ASTM specifications for Grade 100 steel bars.

Purpose. To evaluate the performance of high-strength reinforcement in flexure-critical shear walls subjected to inelastic displacement reversals. Current ACI 318-14 provisions limit shear, flexural, and axial reinforcement to Grade 60. The study will evaluate the applicability of current provisions to walls reinforced with high-strength reinforcing bars subjected to combined shear, flexural, and axial loads. Provisions could be extended for high-strength reinforcement, or reworked with additional variables to accommodate differing behavior.

Details of Study. This study consists of experimental testing of special flexure-critical walls for shear, *P-M* interaction, confinement, and coupling beam behavior. Wall specimens should include confinement details at boundary elements consistent with findings from the initial column tests of Section 4.9.3. Testing of reinforced concrete coupling beams is included, as these elements are used in conjunction with special structural walls to create coupled structural wall systems. Design of the specimens should be in compliance with ACI 318 requirements for special structural walls, as well as the additional detailing requirements identified in NIST GCR 14-917-30.

Shear. Perform cyclic tests of special shear walls with aspect ratios (height-to-length) not less than 3:1. The primary variable in this study is the amount of flexural reinforcement needed to induce shear stresses in the range of $4\sqrt{f'_c}$ to $6\sqrt{f'_c}$. Additional variables could include concrete compressive strength and the presence of wall flanges. This study should explore the validity of Equation 21-7 in ACI 318-11 when using high-strength reinforcement.

P-M Interaction. Perform cyclic lateral loading for special shear walls with aspect ratios (height-to-length) not less than 3:1. The primary variable in this study is vertical reinforcement ratio (e.g., ranging from 0.0025 to 0.03). Axial load ratio, $P/(A_g f'_c)$, could also be varied (e.g., ranging from 0.05 to 0.30). The goal is to confirm the validity of standard *P-M* interaction equations for high-strength reinforcement. Additionally, Grade 60 specimens should be used as benchmarks for evaluating *P-M* ductility and hysteretic energy absorption, to which high-strength reinforcement specimens will be compared. The resulting ductility should also be compared to that intended for special walls in ACI 318 and ASCE/SEI 7.

Confinement. As in previous ACI 318 provisions, confinement for special boundary elements can be considered similar to requirements for columns in special moment frames, avoiding the need for supplementary testing. Outside of the requirements for special boundary elements, current provisions require ties for portions of walls with vertical reinforcement ratios greater than $400/f_y$ to prevent bar buckling. This trigger needs to be validated with physical testing using vertical bars stronger than the current limit of Grade 60. These tests could also be conducted using column specimens.

Coupling Beams. Perform cyclic testing of coupling beams (with and without diagonal reinforcement) subjected to large displacement reversals. This study should include shear stresses up to $8.5\sqrt{f_c'}$ for diagonally-reinforced beams, and $6\sqrt{f_c'}$ for coupling beams reinforced using the provisions that apply to special moment frames. Span-to-depth ratios between 1 and 3 should be considered. The goal is to explore the validity of the current V_n equation using high-strength reinforcement, as well as the potential need for additional confinement in coupling beams with high-strength reinforcement.

For budgeting purposes, the number of test specimens in this test program is anticipated to be on the order of 10 walls (six coupling beam tests and two coupled wall tests). Specimens should be distributed across Grade 80, Grade 100, and Grade 120 reinforcement, optimizing coverage of the remaining parameters.

Team. Three teams (separate research facilities assumed), two teams consisting of one researcher, one full-time graduate student (three years), and one part-time laboratory technician to assist, and one team consisting of one researcher, two full-time graduate students (three years), and one part-time laboratory technician to assist, along with one part-time engineering practitioner to provide structural advice, consultation, and design assistance on each team.

Timeline. Approximately 4 years, assuming 3 years for each team to complete their assigned tests in parallel, but with a staggered schedule.

4.9.11 Performance of Wall Systems

An analytical study on the collapse resistance of structural wall systems constructed using high-strength reinforcement.

Purpose. To assess the collapse resistance of flexure-critical and shear-critical structural walls reinforced with high-strength reinforcement. This study is needed to demonstrate that structural wall systems incorporating the use of high-strength reinforcement are capable of providing a level of safety that is consistent with the seismic performance intended in ASCE/SEI 7-10, and consistent with the performance of seismic-force-resisting systems that have traditionally been designed and constructed using Grade 60 reinforcement. It is intended to address questions regarding overall system behavior that are expected from the National Earthquake Hazard Reduction Program (NEHRP) Provisions Update Committee (PUC), and the ASCE/SEI 7 Seismic Code Committee, when considering reference to future editions of ACI 318 specifically incorporating the use of high-strength reinforcement.

Details of Study. This study consists of nonlinear response history analyses on a series of shear wall buildings using the methodology described in FEMA P-695, *Quantification of Building Seismic Performance Factors* (FEMA, 2009). Model parameters used in this study should be calibrated based on results from component and system tests outlined in Sections 4.9.8, 4.9.9, and 4.9.10.

FEMA P-695 analyses should be performed on prototype buildings with structural walls designed using high-strength reinforcement. Models that were developed in prior studies considering Grade 60 reinforcement (FEMA, 2009; NIST, 2010; and NIST, 2012), could be adapted for use with high-strength reinforcement. Building archetypes from prior studies, including medium- and high-rise walls, should be redesigned with Grade 80, Grade 100, and Grade 120 reinforcement. Buildings should be sited in regions of moderate and high seismicity

Results will be used to compare the collapse resistance of structural wall buildings with high-strength reinforcement to the criteria defined in FEMA P-695. If acceptable collapse resistance is not achieved, the study should identify design measures, material properties, or detailing requirements that would serve to improve collapse behavior. Such measures could be used as the basis for the development of design and detailing rules for structural walls with high-strength reinforcement that would: (1) provide acceptable resistance to collapse; and (2) be suitable for inclusion in ACI 318.

Team. One researcher, one full-time graduate student, and one part-time engineering practitioner to provide advice, consultation, and design assistance.

Timeline. Approximately 2 years.

4.10 Engineering Design Studies

Studies related to trial engineering designs are summarized in Table 4-8.

Table 4-8 Studies Related to Trial Engineering Designs

Reference Section	*Research or Engineering Study*	*Estimated Timeline*
4.10.1	Trial Engineering Designs for Use of Grade 80 Reinforcement in Special Seismic Systems	1 year
4.10.2	Trial Engineering Designs for Use of Grade 100 Reinforcement in General Applications	1 year
4.10.3	Trial Engineering Designs for Use of Grade 100 Reinforcement in Special Seismic Systems	1 year

4.10.1 Trial Engineering Designs for Use of Grade 80 Reinforcement in Special Seismic Systems

An engineering study to develop trial designs of example structures that incorporate and fully utilize Grade 80 high-strength reinforcement in special seismic systems.

Purpose. To: (1) evaluate critical or controlling design issues that are encountered with the use of high-strength reinforcement; (2) provide a point of comparison relative to designs using Grade 60 reinforcement; and (3) evaluate the effects of changes in design requirements on the resulting structural design.

Details of Study. This study consists of trial engineering designs to evaluate and illustrate the influence of incorporating high-strength reinforcement into special seismic systems. It is intended to investigate how the combined effects of new strength, serviceability, and detailing requirements control the resulting reinforced concrete section designs. It is expected that such information will be of interest to ACI Committee 318 in the adoption of revised code requirements.

Two example structures in the five- to ten-story range are recommended for study. As a baseline for comparison, each structure should be designed using Grade 60 reinforcement and the requirements of ACI 318-14. To incrementally study the effects of high-strength reinforcement and the interaction of new design provisions, each structure should be redesigned as follows:

- Using Grade 80 reinforcement and the requirements of ACI 318-14, ignoring all limits restricting the strength of reinforcement (f_y and f_{yt}).

- Using Grade 80 reinforcement and any new design requirements for high-strength reinforcement developed based on research outlined in this *Roadmap*.

The trial designs should investigate the following:

- Design requirements that control the resulting design.
- Code limits, if any, that control the resulting design.
- Differences in controlling factors that are keyed to the use of high-strength reinforcement.
- Differences in member size and reinforcing requirements (e.g., cross-sectional area and details of reinforcement).

Team. Two teams (separate engineering firms assumed), each consisting of one lead engineering practitioner and one part-time staff engineer to assist, and one researcher to provide advice and consultation on results and interpretation of recent and ongoing tests.

Timeline. Approximately 1 year.

4.10.2 Trial Engineering Designs for Use of Grade 100 Reinforcement in General Applications

An engineering study to develop trial designs of example structures that incorporate and fully utilize Grade 100 (or higher) high-strength reinforcement in gravity, wind, and ordinary seismic systems.

Purpose. To: (1) evaluate critical or controlling design issues that are encountered with the use of high-strength reinforcement; (2) provide a point of comparison relative to designs using Grade 60 and Grade 80 reinforcement; and (3) evaluate the effects of changes in design requirements on the resulting structural design.

Details of Study. This study consists of trial engineering designs to evaluate and illustrate the influence of incorporating high-strength reinforcement into gravity, wind, and ordinary seismic systems. It is intended to investigate how the combined effects of any additional strength, serviceability, and detailing requirements for Grade 100 (or higher) reinforcement control the resulting reinforced concrete sections. It is expected that such information will be of interest to ACI Committee 318 in the adoption of revised code requirements.

Two example structures in the five- to ten-story range are recommended for study. Structures designed under Section 4.10.1 using Grade 60 reinforcement and the requirements of ACI 318-14 can be used as a baseline for comparison. To incrementally study the effects of Grade 100 (or higher) high-strength reinforcement and the interaction of new design provisions, each structure should be redesigned as follows:

- Using Grade 100 (or higher) reinforcement and the requirements of ACI 318-14, ignoring all limits restricting the strength of reinforcement (f_y and f_{yt}).
- Using Grade 100 (or higher) reinforcement and any new design requirements for high-strength reinforcement developed based on research outlined in this *Roadmap*.

The trial designs should investigate the following:

- Design requirements that control the resulting design.
- Code limits, if any, that control the resulting design.
- Differences in controlling factors that are keyed to the use of high-strength reinforcement.
- Differences in member size and reinforcing requirements (e.g., cross-sectional area and details of reinforcement).

Team. Two teams (separate engineering firms assumed), each consisting of one lead engineering practitioner and one part-time staff engineer to assist, and one researcher to provide advice and consultation on results and interpretation of recent and ongoing tests.

Timeline. Approximately 1 year.

4.10.3 Trial Engineering Designs for Use of Grade 100 Reinforcement in Special Seismic Systems

An engineering study to develop trial designs of example structures that incorporate and fully utilize Grade 100 (or higher) high-strength reinforcement in special seismic systems.

Purpose. To: (1) evaluate critical or controlling design issues that are encountered with the use of high-strength reinforcement; (2) provide a point of comparison relative to designs using Grade 60 and Grade 80 reinforcement; and (3) evaluate the effects of changes in design requirements on the resulting structural design.

Details of Study. This study consists of trial engineering designs to evaluate and illustrate the influence of incorporating high-strength reinforcement into special seismic systems. It is intended to investigate how the combined effects of any additional strength, serviceability, and detailing requirements for Grade 100 (or higher) reinforcement control the resulting reinforced concrete sections. It is expected that such information will be of interest to ACI Committee 318 in the adoption of revised code requirements.

Two example structures in the five- to ten-story range are recommended for study. Structures designed under Section 4.10.1 using Grade 60 reinforcement and the requirements of ACI 318-14 can be used as a baseline for comparison. To incrementally study the effects of Grade 100 (or higher) high-strength reinforcement and the interaction of new design provisions, each structure should be redesigned as follows:

- Using Grade 100 (or higher) reinforcement and the requirements of ACI 318-14, ignoring all limits restricting the strength of reinforcement (f_y and f_{yt}).
- Using Grade 100 (or higher) reinforcement and any new design requirements for high-strength reinforcement developed based on research outlined in this *Roadmap*.

The trial designs should investigate the following:

- Design requirements that control the resulting design.
- Code limits, if any, that control the resulting design.
- Differences in controlling factors that are keyed to the use of high-strength reinforcement.
- Differences in member size and reinforcing requirements (e.g., cross-sectional area and details of reinforcement).

Team. Two teams (separate engineering firms assumed), each consisting of one lead engineering practitioner and one part-time staff engineer to assist, and one researcher to provide advice and consultation on results and interpretation of recent and ongoing tests.

Timeline. Approximately 1 year.

Chapter 5
Program Recommendations

This chapter summarizes recommended research and engineering studies, provides estimated budget requirements, identifies possible priorities, and provides recommendations for the implementation of a program to encourage adoption of high-strength reinforcement into future editions of ACI 318.

5.1 Summary of Program

In this *Roadmap*, a series of research and engineering studies are proposed to address key production, fabrication, and engineering issues related to the use of high-strength reinforcement in reinforced concrete construction. The proposed research and engineering studies are intended to: (1) further investigate the use of high-strength reinforcement; and (2) support the development of code change proposals for ACI 318 to allow the general use of reinforcement in excess of Grade 60. The list of research and engineering studies is summarized in Table 5-1. Detailed information on the purpose of each study, the scope of recommended testing, the anticipated team, and estimated timeline is provided in Chapter 4.

The overall program is necessarily comprehensive. Together, the proposed research and engineering studies cover the major areas of production and fabrication, strength of members, serviceability, reinforcing limits, detailing, analysis, and earthquake resistance, and affect nearly every chapter of ACI 318.

5.2 Estimated Budget Requirements

The total estimated budget required to implement the program outlined in this *Roadmap* is summarized in Table 5-1. In order to comprehensively investigate all parameters of interest, the number of required test specimens quickly rises to levels that are impractical to implement. As a result, the number of test specimens, and the number of parameters investigated, has been intentionally limited in each study. The resulting recommendations are a compromise between the need to cover variation in important controlling parameters, and what can reasonably be accomplished in the environment of a traditional university research program.

Table 5-1 Summary of Proposed Research and Engineering Studies

Reference Section	*Research or Engineering Study*	*Estimated Budget*
4.3	**Bar Production and Specification**	
4.3.1	Mechanical Properties of Recent Heats of High Strength Reinforcement	$100,000
4.3.2	Detailed Mechanical Property Tests of Grade 100 and Grade 120 Reinforcement	$439,000
4.4	**Strength of Members**	
4.4.1	Flexural Strength and Tensile Strain Limits	$149,000
4.4.2	Required Deflection of Flexural Members Subjected to Gravity Loads	$518,000
4.4.3	Column Strength	$26,000
4.4.4	Tension Regions of Shells and Folded Plates	$834,000
4.4.5.1	One-Way Shear in Beams without Shear Reinforcement	$692,000
4.4.5.2	Two-Way Shear in Slabs without Shear Reinforcement	$716,000
4.4.6	Shear Strength of Beams with Shear Reinforcement	$716,000
4.4.7	Shear-Friction	$384,000
4.4.8	High-Cycle Elastic Fatigue of High-Strength Reinforcing Bars	$550,000
4.5	**Serviceability**	
4.5.1	Deflection of Flexural Members	$611,000
4.5.2	Crack Control of Flexural Members	$341,000
4.6	**Reinforcing Limits**	
4.6.1	Minimum Reinforcement Ratio for Beams	$43,000
4.6.2	Minimum Reinforcement Ratios for Slabs and Footings	$43,000
4.7	**Detailing of Members**	
4.7.1	Development and Splice Lengths	$738,000
4.7.2	Hooked Bar Development Length	$988,000
4.7.3	Headed Bar Development Length	$623,000
4.8	**General Considerations for Analysis**	
4.8.1	Flexural Stiffness	$209,000
4.8.2	Effective Stiffness for Column Slenderness	$383,000
4.8.3	Moment Redistribution	$209,000
4.9	**Earthquake-Resistant Structures**	
4.9.1	Moment-Curvature and Rotational Capacity	$151,000
4.9.2	Factor for Estimating Expected Flexural Strength	$197,000
4.9.3	Cyclically Loaded Beams and Columns – Initial Tests and Analytical Studies	$749,000
4.9.4.1	Cyclically Loaded Beams	$1,654,000
4.9.4.2	Cyclically Loaded Columns	$1,734,000
4.9.4.3	Cyclically Loaded Interior Joints	$1,541,000

Table 5-1 Summary of Proposed Research and Engineering Studies (continued)

Reference Section	*Research or Engineering Study*	*Estimated Budget*
4.9.4.4	Cyclically Loaded Exterior Joints	$1,429,000
4.9.4.5	Two-Way Shear in Slab-Column Intermediate Moment Frames	$809,000
4.9.5	Performance of Moment Frame Systems	$265,000
4.9.6	Multi-Bay, Multi-Story Frames	$1,040,000
4.9.7	Ordinary Flexure-Critical Walls	$1,226,000
4.9.8	Special and Ordinary Shear-Critical Walls	$3,105,000
4.9.9	Special Flexure-Critical Walls – Initial Tests	$591,000
4.9.10	Special Flexure-Critical Walls	$2,109,000
4.9.11	Performance of Wall Systems	$265,000
4.10	**Engineering Design Studies**	
4.10.1	Trial Engineering Designs for Use of Grade 80 Reinforcement in Special Seismic Systems	$268,000
4.10.2	Trial Engineering Designs for Use of Grade 100 Reinforcement in General Applications	$268,000
4.10.3	Trial Engineering Designs for Use of Grade 100 Reinforcement in Special Seismic Systems	$268,000
	Total Estimated Program Budget	**$26,981,000**

Estimated budget requirements by program area (production and fabrication, strength of members, serviceability, reinforcing limits, detailing, analysis, and earthquake resistance) are summarized in Table 5-2.

Table 5-2 Estimated Budget Requirements by Program Area

Reference Section	*Study*	*Estimated Budget*
4.3	Bar Production and Specification	$539,000
4.4	Strength of Members	$4,585,000
4.5	Serviceability	$952,000
4.6	Reinforcing Limits	$86,000
4.7	Detailing of Members	$2,349,000
4.8	General Considerations for Analysis	$801,000
4.9	Earthquake-Resistant Structures	$16,865,000
4.10	Engineering Design Studies	$804,000

The effects of high-strength reinforcement on strength, serviceability, detailing, and member behavior are interrelated, and testing in one area can impact the results of testing in another area. In preparing budget estimates, however, individual research and engineering studies have been priced individually. Some consideration has been given to combining studies and using the results from a series of tests in one area to serve the needs of another area. It is anticipated that program recommendations will evolve as implementation progresses, and that additional efficiencies will be realized as test results become available, lessons are learned, and the state of knowledge improves on which parameters are most important.

5.2.1 Budget Assumptions

Budgeting assumptions have been based on the traditional university research program model, which consists of a lead researcher assisted by one or more graduate students performing the work. In this model, the lead researcher serves as a part-time mentor and advisor, and the graduate students work essentially full-time in fabricating and testing the specimens, processing results, performing necessary analytical studies, and extracting conclusions. Additional participants in this model include part-time laboratory technicians assisting in the fabrication and testing of specimens, and engineering practitioners advising on structural design and construction issues.

As the estimates for individual research and engineering studies were developed, the timing of a typical degree cycle (Ph.D. or M.S. program), and the amount of work (fabrication and testing of specimens) that could reasonably be accomplished at a typical research facility within a typical degree cycle, were considered. The following basic budget categories were included in the estimates:

- Salary (lead researcher and graduate students)
- Tuition (graduate students)
- Travel (all participants)
- Specimen material and fabrication costs
- Laboratory costs
- Laboratory labor costs (technicians)
- Consultant costs (engineering advisors)

Unit direct costs for the above budget categories were based on typical (yet conservative) values taken from major research institutions across the United States. In addition to itemized direct costs, budget estimates included an allowance for typical indirect costs associated with the university research program model. These included an assumed fringe benefit rate of 1.15 on salaries, and an overhead rate of 1.6 on direct charges (except tuition).

5.3 Priority and Schedule Recommendations

The recommended program of research and engineering study is ambitious, and practical implementation will require prioritization. Practical considerations for prioritization include the need to define material properties for high-strength reinforcement, the timing of future code update cycles, the time it takes to fabricate and test large numbers of specimens, the relative costs, and the potential benefits associated with the use of high-strength reinforcement in selected areas of reinforced concrete design and construction (i.e., where it is most likely to be used, and where it provides the most advantageous result). Based on the above considerations, the objectives identified in Table 5-3 are recommended as a basis for prioritization of the program.

Table 5-3 Recommended Priorities Based on Implementation Objectives and Target Milestone Dates

Priority Level	*Objective*	*Target Milestone*	*Approximate Timeline*
1	Revise ASTM A615 to include Grade 100 reinforcement	2016 Interim Provisions for ACI 318	Research completed by December 2015
2	Modify ACI 318 to allow the use of ASTM A706 Grade 80 reinforcement in special seismic systems	2016 Interim Provisions for ACI 318	Research completed by December 2015
3	Modify ACI 318 to allow the use of ASTM A615 Grade 100 reinforcement in general applications (gravity, wind, and ordinary seismic systems)	2019 edition of ACI 318	Research completed by December 2017
4	Develop a new ASTM specification for Grade 100 reinforcement for use in special seismic systems	2022 (or later) edition of ACI 318	Research completed by December 2020 (or later)
5	Modify ACI 318 to allow the use of Grade 100 reinforcement in special seismic systems	2022 (or later) edition of ACI 318	Research completed by December 2020 (or later)

To date, ACI Committee 318 has operated on a three-year code update cycle, but the schedule for future update cycles is in transition. At present, ACI Committee 318 is considering the publication of *Interim Provisions* in 2016, followed by a 2019 edition of ACI 318. Beyond the 2019 edition, it is unclear if future updates will occur on a three-year, five-year, or six-year cycle.

The milestones listed in Table 5-3 represent an idealized timeline that targets future editions of ACI 318 in the relative near-term, assuming the availability of adequate funding, sufficient production and availability of high-strength reinforcing materials, and the availability of appropriately equipped laboratory facilities capable of performing the necessary testing. Individual research or engineering studies contributing to each objective at each priority level are summarized in the sections that follow.

5.3.1 Priority Level 1 – Revise ASTM A615 to Include Grade 100 Reinforcement

Revising ASTM A615 to include Grade 100 reinforcement is considered a Priority Level 1 objective because the existence of a specification for the material will allow its use in certain applications. Demand for the material will then provide an opportunity for material producers to refine their metallurgy and manufacturing processes to better meet requirements (e.g., ductility and tensile-to-yield strength ratios) related to engineering needs associated with the development of Grade 100 reinforcement for seismic and other applications. Studies related to Priority Level 1 objectives are summarized in Table 5-4.

Table 5-4 Priority Level 1 – Revise ASTM A615 to Include Grade 100 Reinforcement

Reference Section	*Research or Engineering Study*	*Estimated Budget*
4.3.1	Mechanical Properties of Recent Heats of High Strength Reinforcement	$100,000
	Total Estimated Budget – Priority Level 1	**$100,000**

The ASTM committee responsible for specification A615 has begun the process of adding Grade 100 to its specification. Data from recent heats has already been collected and analyzed. Test results from additional heats are required and confirmation testing by an independent laboratory is needed.

The first version of ASTM A615 to include Grade 100 reinforcement may require adjustment before ACI Committee 318 is willing to adopt it. Studies to facilitate adoption of ASTM A615 Grade 100 reinforcement by ACI Committee 318 are included under Priority Level 3.

5.3.2 Priority Level 2 – Modify ACI 318 to Allow the use of ASTM A706 Grade 80 Reinforcement in Special Seismic Systems

Modifying ACI 318 to allow use ASTM A706 Grade 80 reinforcement in special seismic systems is considered a Priority Level 2 objective because there is a strong demand for use of high-strength reinforcement in this area, and the research and engineering study to make a change to Grade 80 reinforcement is relatively limited. Studies related to Priority Level 2 objectives are summarized in Table 5-5. To meet the target milestone of the 2016 *Interim Provisions* of ACI 318, testing associated with these studies would need to be completed by December 2015.

Table 5-5 Priority Level 2 – Modify ACI 318 to Allow the use of ASTM A706 Grade 80 Reinforcement in Special Seismic Systems

Reference Section	*Research or Engineering Study*	*Estimated Budget*[(1)]
4.9.1	Moment-Curvature and Rotational Capacity (Grade 80 reinforcement only)	$50,000
4.9.4.3	Cyclically Loaded Interior Joints (Grade 80 reinforcement only)	$509,000
4.9.5	Performance of Moment Frame Systems (Grade 80 reinforcement only)	$88,000
4.9.10	Special Flexure-Critical Walls (Grade 80 reinforcement only)	$696,000
4.9.11	Performance of Wall Systems (Grade 80 reinforcement only)	$88,000
4.10.1	Trial Engineering Designs for Use of Grade 80 Reinforcement in Special Seismic Systems	$268,000
	Total Estimated Budget – Priority Level 2	**$1,699,000**

Notes: (1) Budgeted amounts have been prorated for the conduct of tests or engineering studies on Grade 80 reinforcement only.

Studies listed in Table 5-5 address knowledge gaps related to members reinforced with Grade 80 reinforcement. These include the performance of moment frames and structural wall systems, and the required depth of interior joints in special moment frames. These studies also include investigation of configurations with higher strength reinforcement (i.e., Grade 100 and Grade 120 reinforcement). To achieve Priority Level 2 objectives, only the subset of work related to Grade 80 reinforcement need to be completed within this timeline. Tests on higher grade reinforcement can be deferred to Priority Level 5.

5.3.3 Priority Level 3 – Modify ACI 318 to Allow the use of ASTM A615 Grade 100 Reinforcement in General Applications (Gravity, Wind, and Ordinary Seismic Systems)

Modifying ACI 318 to allow use ASTM A615 Grade 100 reinforcement for general application in gravity, wind, and ordinary seismic systems is considered a Priority Level 3 objective because the level of research and engineering study required to validate its use is more significant than can be realistically be accomplished in a shorter timeframe. Studies related to Priority Level 3 objectives have been further subdivided into relative priorities of high, moderate, and low, as summarized in Table 5-6.

It should be noted that use of Grade 100 reinforcement may not be appropriate in all applications. Elements with stringent serviceability requirements (e.g., slabs and beams) may experience limited benefit from the use of high-strength reinforcement, because serviceability concerns are expected to control, while elements without stringent serviceability requirements (e.g., foundations, columns, and walls) may potentially see greater benefit.

Table 5-6 Priority Level 3 – Modify ACI 318 to Allow the use of ASTM A615 Grade 100 Reinforcement in General Applications (Gravity, Wind, and Ordinary Seismic Systems)

Reference Section	*Research or Engineering Study*	*Estimated Budget*
	High (relative priority)	
4.3.2	Detailed Mechanical Property Tests of Grade 100 and Grade 120 Reinforcement (1)	$219,000
4.4.1	Flexural Strength and Tensile Strain Limits	$149,000
4.4.2	Required Deflection of Flexural Members Subjected to Gravity Loads	$518,000
4.4.5.1	One-Way Shear in Beams without Shear Reinforcement	$692,000
4.4.5.2	Two-Way Shear in Slabs without Shear Reinforcement	$716,000
4.4.7	Shear-Friction	$384,000
4.6.2	Minimum Reinforcement Ratios for Slabs and Footings	$43,000
4.7.1	Development and Splice Lengths	$738,000
4.7.2	Hooked Bar Development Length	$988,000
4.7.3	Headed Bar Development Length	$623,000
4.9.7	Ordinary Flexure-Critical Walls (1)	$1,226,000
4.10.2	Trial Engineering Designs for Use of Grade 100 Reinforcement in General Applications	$268,000
	Moderate (relative priority)	
4.4.3	Column Strength	$26,000
4.5.1	Deflection of Flexural Members	$611,000
4.5.2	Crack Control of Flexural Members	$341,000
4.6.1	Minimum Reinforcement Ratio for Beams	$43,000
4.8.1	Flexural Stiffness (1)	$105,000
4.8.2	Effective Stiffness for Column Slenderness	$383,000
	Low (relative priority)	
4.4.6	Shear Strength of Beams with Shear Reinforcement	$716,000
4.8.3	Moment Redistribution	$209,000
4.9.8	Special and Ordinary Shear-Critical Walls (1)	$3,105,000
	Total Estimated Budget – Priority Level 3	**$9,937,000**

Notes: (1) Budgeted amount has been prorated to account for the conduct of tests on Grade 100 reinforcement only.

5.3.4 Priority Level 4 – Develop a new ASTM specification for Grade 100 Reinforcement for use in Special Seismic Systems

Development of a new ASTM specification that includes Grade 100 reinforcement for use in special seismic systems is considered a Priority Level 4 objective. Prior to developing such a specification, research that establishes the required mechanical properties, and what elongation and tensile-to-yield strength ratios can be achieved for this grade of reinforcement, must be completed. Studies related to Priority Level 4 objectives are summarized in Table 5-7.

Table 5-7 Priority Level 4 – Develop a new ASTM Specification for Grade 100 Reinforcement for use in Special Seismic Systems

Reference Section	*Research or Engineering Study*	*Estimated Budget*
4.3.2	Detailed Mechanical Property Tests of Grade 100 and Grade 120 Reinforcement (1)	\$219,000
4.9.3	Cyclically Loaded Beams and Columns – Initial Tests and Analytical Studies	\$749,000
4.9.9	Special Flexure-Critical Walls – Initial Tests	\$591,000
	Total Estimated Budget – Priority Level 4	**\$1,559,000**

Notes: (1) Budgeted amount has been prorated to account for the portion of tests conducted under Priority Level 3.

5.3.5 Priority Level 5 – Modify ACI 318 to Allow the use of Grade 100 Reinforcement in Special Seismic Systems

Modifying ACI 318 to allow the use of Grade 100 reinforcement in special seismic systems is considered a Priority Level 5 objective. It is anticipated that there will high demand for the use of Grade 100 reinforcement in seismic applications; however, the level of effort needed to justify its use is significantly greater than for other priorities. Studies related to Priority Level 5 objectives have been further subdivided into relative priorities of high and moderate, as summarized in Table 5-8.

5.4 Other Conclusions and Recommendations

Several issues raised during the preparation of this *Roadmap* were judged capable of resulting in a potential code change without the need for additional research or engineering study. Some issues were judged as not needing further action or consideration, either in the form of additional research or code change. Finally, some issues were judged as needing further research, but the resulting studies were not considered critical for implementation of high-strength reinforcement in ACI 318. These are included here for reference, and might warrant additional study at a later time.

Table 5-8 Priority Level 5 – Modify ACI 318 to Allow the use of Grade 100 Reinforcement in Special Seismic Systems

Reference Section	*Research or Engineering Study*	*Estimated Budget*
	High (relative priority)	
4.9.8	Special and Ordinary Shear-Critical Walls (1)	$1,552,000
4.9.10	Special Flexure-Critical Walls (2)	$1,413,000
4.9.11	Performance of Wall Systems (2)	$178,000
	Moderate (relative priority)	
4.8.1	Flexural Stiffness (1)	$105,000
4.9.1	Moment-Curvature and Rotational Capacity (2)	$101,000
4.9.2	Factor for Estimating Expected Flexural Strength	$197,000
4.9.4.1	Cyclically Loaded Beams	$1,654,000
4.9.4.2	Cyclically Loaded Columns	$1,734,000
4.9.4.3	Cyclically Loaded Interior Joints (2)	$1,033,000
4.9.4.4	Cyclically Loaded Exterior Joints	$1,429,000
4.9.4.5	Two-Way Shear in Slab-Column Intermediate Moment Frames	$809,000
4.9.5	Performance of Moment Frame Systems (2)	$178,000
4.9.6	Multi-Bay, Multi-Story Frames	$1,040,000
4.9.7	Ordinary Flexure-Critical Walls (1)	$613,000
4.10.3	Trial Engineering Designs for Use of Grade 100 Reinforcement in Special Seismic Systems	$268,000
	Total Estimated Budget – Priority Level 5	**$12,304,000**

Notes: (1) Budgeted amount has been prorated to account for the portion of tests or engineering studies conducted under Priority Level 3.

(2) Budgeted amount has been prorated to account for the portion of tests or engineering studies conducted under Priority Level 2.

5.4.1 Potential Code Changes that can be Implemented without Additional Research

Issues that were judged capable of resulting in a potential code change without the need for additional research or engineering study are summarized in Table 5-9. These issues are applicable to Priority Level 3 objectives to allow the use of Grade 100 reinforcement for general application in gravity, wind, and ordinary seismic systems.

In each case, it was felt that code change proposals could be developed, and approved, based on the consensus opinion of ACI Committee 318 based on currently available information. To facilitate adoption, it may be useful to develop a published source (e.g., a summary paper in *Concrete International* or journal article) that can serve as a reference for the proposed change.

Table 5-9	Potential Code Changes that can be Implemented without Additional Research	
Reference Section	*Issue*	*Recommended Action*
3.4.2.1	Spacing of transverse reinforcement in members resisting gravity and wind loads, or seismic loads in ordinary seismic-force-resisting systems	ACI Committee 318 to determine required spacing of transverse reinforcement based on existing tests using Grade 60 reinforcement and consensus opinion
3.4.4.4	Mechanical splices that develop the actual tensile strength of high-strength reinforcement	ACI Committee 318 to determine the need for such a mechanical splice based on current information and consensus opinion
3.4.6	Horizontal support of offset column reinforcement	ACI Committee 318 to determine the need for additional horizontal support based on current information and consensus opinion
3.6.6	High-strength reinforcement in diaphragms	ACI Committee 318 to determine applicability to diaphragms based on current information and consensus opinion

5.4.2 Issues Not Requiring Further Action or Code Change

Several issues considered during the preparation of this *Roadmap* were judged as not warranting further action or a code change. These findings should be presented to ACI Committee 318 for consideration and confirmation, but no further action is expected or recommended. Issues not requiring further action or code change are summarized in Table 5-10.

Table 5-10	Issues Not Requiring Further Action or Code Change	
Reference Section	*Issue*	*Recommended Action*
3.1.1.2	Longitudinal bar maximum useable compressive stress limited to 80 ksi	None
3.1.2.4	Minimum reinforcement ratio for deep beams	None
3.1.3	Strut-and-tie modeling using high-strength reinforcement	None
3.1.4.1	Minimum concrete strength for use with high-strength reinforcement	None
3.1.4.3	Bonded reinforcement ratios for members with unbonded post-tensioning	None
3.3.3	Minimum reinforcement ratio for columns	None
3.3.4	Minimum reinforcement ratio for walls	None
3.4.3	Headed reinforcement head size and attachment to bars (non-seismic applications)	None
3.4.5.1	Bar extensions in one-way slabs	None
3.4.5.2	Bar extensions in two-way slabs	None
3.4.7	Cover for Fire Protection	None

5.4.3 Other Potential Studies

Two studies were judged not essential for the adoption of high-strength reinforcement into ACI 318 for gravity, wind, or seismic applications. These are listed in Table 5-11 for reference, and might warrant additional study at a later time, or might be of interest to other funding agencies.

Table 5-11 Other Potential Studies

Reference Section	*Research or Engineering Study*	*Estimated Budget*
4.4.4	Tension Regions of Shells and Folded Plates	$834,000
4.4.8	High-Cycle Elastic Fatigue of High-Strength Reinforcing Bars	$550,000
	Total Estimated Budget – Other Potential Studies	**$1,384,000**

5.5 Implementation Recommendations

Implementation of the program outlined in this *Roadmap* will likely involve the following activities:

- Identification of key collaborators and potential funding partners
- Technical oversight, including ongoing monitoring of results and adjustment of priorities
- Technical synthesis and development of products
- Advocacy and adoption into codes and standards

The path from research results to code change is not always direct. Given the magnitude of the program, and the complexity and extent of code changes needed to incorporate the use of high-strength reinforcement into ACI 318, a plan should be implemented to manage the research effort, coordinate between studies, continually monitor results, and adjust the scope and priorities of future studies as new information becomes available.

5.5.1 Key Collaborators

Use of high-strength reinforcement in reinforced concrete design and construction has attracted the attention of a number of stakeholders who are potential collaborators on the implementation of this *Roadmap*. These include material producers, industry associations, and governmental agencies, some of which will be providers of necessary information, or sources of supplemental funding.

Potential key collaborators include the American Concrete Institute (ACI), Portland Cement Association (PCA), and Concrete Reinforcing Steel

Institute (CRSI). Potential Federal partners include the National Institute for Standards and Technology (NIST), Federal Emergency Management Agency (FEMA), National Science Foundation (NSF), and Department of Energy (DOE).

In some cases, logical partners can be identified by the nature of the studies. One such example is fatigue testing of beams high-strength reinforcement, in which case logical partners might be the Transportation Research Board (TRB) of the National Academies, and the Federal Highway Administration (FHWA).

Other potential partners include industry groups that are likely to benefit from the use of high-strength reinforcement, such as the Electric Power Research Institute (EPRI) and Nuclear Energy Institute (NEI).

5.5.2 Technical Oversight

The purpose of a technical oversight function is to make sure that studies are properly focused and implemented as efficiently as possible. It also provides a mechanism to adjust the scope and priorities of future tests based on results of tests completed to date.

Technical oversight can be used to: (1) monitor how each study fits within the overall program; (2) provide advice regarding technical aspects of the testing, including specimen design, parametric coverage of variables, loading protocols, and necessary data; (3) evaluate whether a single series of tests can be used to addresses more than one issue; (4) recommend priorities based on the timing of future ACI 318 update cycles and relative impacts to design and construction; (5) help ensure that research stays on target and will effectively lead to code changes; (6) provide quality assurance reviews of test results for accuracy and reasonableness of conclusions; and (7) provide recommendations for modifications to the program as intermediate results are obtained and reviewed.

Ideally, technical oversight should be performed by a panel consisting of engineering practitioners and researchers, with representation from ACI Committee 318 and relevant subcommittees.

5.5.3 Technical Synthesis

Given the number and complexity of the research and engineering studies outlined in this *Roadmap*, and the extent of code changes necessary to incorporate the use of high-strength reinforcement into ACI 318, there is a need to process information from individual research studies into a coherent set of results (i.e., synthesis). A technical synthesis function would serve to bring consistency to the interpretation of results, and would use the resulting test data to formulate coordinated recommendations to be sent to ACI Committee 318 for consideration.

A technical synthesis strategy has been used in other major research and development programs, and was instrumental in the development of new steel moment frame guidelines following the 1994 Northridge earthquake. FEMA 350, *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings* (FEMA, 2000) is one in a series of products developed based on technical synthesis of the results of a multi-year, multi-million dollar experimental research effort funded by FEMA.

Ideally, technical synthesis should be performed by a group of experts on the topic under consideration, including engineering practitioners and researchers, with representation from ACI Committee 318 and relevant subcommittees.

5.5.4 Adoption into Codes and Standards

Changes to ACI 318 can originate from many sources, including engineers not associated with the ACI Committee 318 process, responses from the public review process, and the work of other committees. Most, however, originate from members of ACI Committee 318. Change is initiated with a proposal that includes an explanation of why the change is being proposed, the proposed change to the code provisions and commentary sections, and supporting information. The reason for the proposed change sets the stage with background information, and is generally relatively brief. The code and commentary changes are prepared in strike-through and underline form. Supporting information is often elaborate, especially for highly technical changes, and can be similar to a technical paper. For the research and engineering studies outlined in this *Roadmap*, resulting code change proposals will require substantial supporting information.

Proposals are most often prepared by one or more subcommittee members, but first drafts can come from other sources and assigned to the subcommittee. Once the draft is available, the subcommittee vets the proposal through a balloting process. After the proposal is modified and approved by the subcommittee, it is balloted by the main committee. Ballot results are addressed by the subcommittee and sent back to the main committee for re-balloting. This process can iterate between the main committees and the subcommittee multiple times.

To address code change proposals related to the use of high-strength reinforcement, ACI Committee 318 has formed a subcommittee, designated

ACI 318R. Results from the research and engineering studies outlined in this *Roadmap* can be packaged for use by ACI 318 in multiple ways:

1. Raw data for individual studies can be provided to ACI 318R or other relevant ACI subcommittees, and members can interpret the results and create code change proposals.
2. Data from studies can be processed by individual research teams, and general recommendations for change can be provided.
3. Data from studies can be processed by individual research teams, and formal code change proposals can be provided.
4. Data from studies can be processed by individual research teams, but results across multiple studies are synthesized by a technical synthesis group administered as part of the program.
5. Results from the overall program can be synthesized by a technical synthesis group administered as part of the program and charged with the development of coordinated packages of formal code change proposals.

ACI Committee 318 and its subcommittees are made up of volunteers that could potentially become overwhelmed by a substantial number of complex and interrelated change proposals. For that reason, the approaches identified in options 4 and 5 are likely to be the preferred approach, and will likely speed the process of adoption within ACI Committee 318. In some cases, coordination between studies is not needed, and approaches identified in options 1, 2, or 3 could be used. Of these options, 2 is the most desirable because it will allow for some thought to be given on how to approach the change proposal, and the time to generate general recommendations is less than the time it could take to produce formal code change proposals. The approach in option 3 is the least desirable because of potential delays in considering the information, and lack of coordination with other changes. The approach in option 1 places the greatest burden on ACI Subcommittee 318R and other relevant ACI subcommittees, but it may be necessary in some cases, particularly when code update cycle deadlines are approaching.

References

AASHTO, 2014, *AASHTO LRFD Bridge Design Specifications, Customary U.S. Units, 7th Edition, with 2015 Interim Revisions*, American Association of State Highway and Transportation Officials, Washington, D.C.

ACI, 1936, *Building Regulations for Reinforced Concrete*, ACI 501-36T, ACI Committee 501, American Concrete Institute, Detroit, Michigan.

ACI, 1941, *Building Code Requirements for Reinforced Concrete*, ACI 318-41, ACI Committee 318, American Concrete Institute, Detroit, Michigan.

ACI, 1956, *Building Code Requirements for Reinforced Concrete*, ACI 318-56, ACI Committee 318, American Concrete Institute, Detroit, Michigan.

ACI, 1963, *Building Code Requirements for Reinforced Concrete*, ACI 318-63, ACI Committee 318, American Concrete Institute, Detroit, Michigan.

ACI, 1971a, *Building Code Requirements for Reinforced Concrete*, ACI 318-71, ACI Committee 318, American Concrete Institute, Detroit, Michigan.

ACI, 1971b, *Commentary on Building Code Requirements for Reinforced Concrete (ACI 318-71)*, ACI Committee 318, American Concrete Institute, Detroit, Michigan.

ACI, 1977, *Building Code Requirements for Reinforced Concrete*, ACI 318-77, ACI Committee 318, American Concrete Institute, Detroit, Michigan.

ACI, 1983, *Building Code Requirements for Reinforced Concrete*, ACI 318-83, ACI Committee 318, American Concrete Institute, Detroit, Michigan.

ACI, 1995, *Control of Deflection in Concrete Structures*, ACI 435R-95, ACI Committee 435, American Concrete Institute, Detroit, Michigan.

ACI, 1997, *Considerations for Design of Concrete Structures Subjected to Fatigue Loading*, ACI 215R-92, ACI Committee 215, American Concrete Institute, Farmington Hills, Michigan.

ACI, 1999, *Building Code Requirements for Structural Concrete and Commentary*, ACI 318-99 and ACI 318R-99, ACI Committee 318, American Concrete Institute, Farmington Hills, Michigan.

ACI, 2002a, *Building Code Requirements for Structural Concrete and Commentary*, ACI 318-02 and ACI 318R-02, ACI Committee 318, American Concrete Institute, Farmington Hills, Michigan.

ACI, 2002b, *Recommendations for Design of Beam-Column Connections in Monolithic Reinforced Concrete Structures*, ACI 352R-02, Joint ACI-ASCE Committee 352, American Concrete Institute, Farmington Hills, Michigan.

ACI, 2003, *Bond and Development of Straight Reinforcing Bars in Tension*, ACI 408R-03, ACI Committee 408, American Concrete Institute, Farmington Hills, Michigan.

ACI, 2006, *Guide for the Design and Construction of Structural Concrete Reinforced with FRP Bars*, ACI 440.1R-06, ACI Committee 440, American Concrete Institute, Farmington Hills, Michigan.

ACI, 2010a, *Design Guide for the Use of ASTM A1035/A1035M Grade 100 Steel Bars for Structural Concrete*, ACI ITG-6R-10, ACI Innovation Task Group 6, American Concrete Institute, Farmington Hills, Michigan.

ACI, 2010b, *Specification for Tolerances for Concrete Construction and Materials and Commentary*, ACI 117-10, ACI Committee 117, American Concrete Institute, Farmington Hills, Michigan.

ACI, 2011, *Building Code Requirements for Structural Concrete and Commentary*, ACI 318-11, ACI Committee 318, American Concrete Institute, Farmington Hills, Michigan.

ACI, 2014a, *Building Code Requirements for Structural Concrete and Commentary*, ACI 318-14, ACI Committee 318, American Concrete Institute, Farmington Hills, Michigan.

ACI, 2014b, *Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary*, ACI 349-13, ACI Committee 349, American Concrete Institute, Farmington Hills, Michigan.

ACI-ASCE Committee 326, 1962, “Shear and diagonal tension,” *ACI Journal Proceedings*, Vol. 59, No. 1, pp. 1-30; No. 2, pp. 277-334; and No. 3, pp. 352-396.

ACI-ASCE Committee 426, 1973, “Shear Strength of Reinforced Concrete Members,” ACI 426R-74, *Journal of the Structural Division*, ASCE, Vol. 99, No. 6, pp. 1148-1157.

Angelakos, D., Bentz, E., and Collins, M., 2001, “Effect of concrete strength and minimum stirrups on shear strength of large members,” *ACI Structural Journal*, Vol. 98, No. 3, pp. 290-300.

Aoyama, H., 2001, *Design of Modern Highrise Reinforced Concrete Structures*, Imperial College Press, London, United Kingdom.

ASCE, 2010, *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7-10, American Society of Civil Engineers, Structural Engineering Institute, Reston, Virginia.

AS/NZS, 2001, *AS/NZS 4671 Steel Reinforcing Materials*, Joint Standards Australia/Standards New Zealand, Wellington, New Zealand and Sydney, Australia.

ASTM, 1959a, *Specification for Deformed Billet Steel Bars for Concrete Reinforcement with 60,000 psi Minimum Yield Point*, ASTM A432, American Society for Testing and Materials, Philadelphia, Pennsylvania.

ASTM, 1959b, *Specification for High-Strength Deformed Billet-Steel Bars for Concrete Reinforcement with 75,000 psi Minimum Yield Strength*, ASTM A431, American Society for Testing and Materials, Philadelphia, Pennsylvania.

ASTM, 1974, *Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement*, ASTM A706, American Society for Testing and Materials, Philadelphia, Pennsylvania.

ASTM, 2009a, *Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement*, ASTM A615-09b, ASTM International, West Conshohocken, Pennsylvania.

ASTM, 2009b, *Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement*, ASTM A706-09b, ASTM International, West Conshohocken, Pennsylvania.

ASTM, 2011*, Standard Specification for Deformed and Plain, Low-Carbon, Chromium, Steel Bars for Concrete Reinforcement*, ASTM A1035, ASTM International, West Conshohocken, Pennsylvania.

ASTM, 2013, *Standard Specification for Headed Steel Bars for Concrete Reinforcement*, ASTM A970 including Annex A1, ASTM International, West Conshohocken, Pennsylvania.

Bang, C.-J., 2014, *Application of High-Strength/Large-Diameter Headed Deformed Bars in Construction of Nuclear Power Plant Structures*, presentation to ACI Committees 349 and 359.

Bilow, D.N., and Kamara, M.E., 2008, "Fire and concrete structures," *Proceedings*, ASCE Structures Congress, Vancouver, Canada.

Birrcher, D.B., Tuchscherer, R.G., Huizinga, M., and Bayrak, O., 2013, "Minimum web reinforcement in deep beams," *ACI Structural Journal*, Vol. 110, No. 2, pp. 297-306.

Bischoff, P.H., 2007, "Deflection calculation of FRP reinforced concrete beams based on modifications to the existing Branson equation," *Journal of Composites for Construction*, Vol. 11, No. 1, pp. 4-14.

Bischoff, P.H., and Scanlon, A., 2007, "Effective moment of inertia for calculating deflections of concrete members containing steel reinforcement and FRP reinforcement," *ACI Structural Journal*, Vol. 104, No. 1, pp. 68-75.

Bournonville, M., Dahnke, J., and Darwin, D., 2004, *Statistical Analysis of the Mechanical Properties and Weight of Reinforcing Bars*, SL Report 04-1, Structural Engineering and Materials Laboratory, University of Kansas, Lawrence, Kansas.

Branson, 1977, *Deformation of Concrete Structures,* McGraw Hill Book Co., Advanced Book Program, New York, New York.

Branson, D.E., and Trost, H., 1982, "Unified Procedures for Predicting the Deflection and Centroidal Axis Location of Partially Cracked Non-Prestressed Members," *ACI Journal Proceedings*, Vol. 79, No. 2, pp. 119-130.

Brown, M., Bayrak, O., and Jirsa, J., 2006, "Design for shear based on loading conditions," *ACI Structural Journal*, Vol. 103, No. 4, pp. 541-550.

Budek, A., Priestley, M., and Lee, C., 2002, "Seismic design of columns with high-strength wire and strand as spiral reinforcement," *ACI Structural Journal*, Vol. 99, No. 5, pp. 660-670.

Chen, M., Pantoli, E., Wang, X., Espino, E., Mintz, S., Conte, J., Hutchinson, T., Marin, C., Meacham, B., Restrepo, J.I., Walsh, K., Englekirk, R., Faghihi, M., and Hoehler, M., 2012, "Design and construction of a full-scale 5-story base isolated building outfitted with nonstructural components for earthquake testing at the UCSD-NEES facility," *Proceedings*, ASCE Structures Congress 2012, pp. 1349-1360.

Choi, W.-S., Park, H.-G. , Chung, L., and Kim, J.-K., 2014, "Experimental study for Class B lap splice of 600 MPa (87 ksi) reinforcing bars," *ACI Structural Journal*, Vol. 111, No. 1, pp. 49-58.

Collins, M., and Kuchma, D., 1999, "How safe are our large, lightly reinforced, concrete beams, slabs, and footings?" *ACI Structural Journal*, Vol. 96, No. 4, pp. 482-490.

CRSI, 2013, Data from an unpublished database of the Concrete Reinforcing Steel Institute, Schaumburg, Illinois.

Davey, R.A., and Blaikie, E.L., 2005, "On the flexural ductility of very lightly reinforced concrete sections," *Proceedings*, 2005 NZSEE Conference, Taupo, New Zealand.

Dazio, A.D.A., Beyer, K., and Bachmann, H., 2009, "Quasi-static cyclic tests and plastic hinge analysis of RC structural walls," *Engineering Structures*, Vol. 31, pp. 1556-1571.

Edwards, W.T., and Gamble, W.L., 1986, "Strength of Grade 60 reinforcing bars after exposure to fire temperatures," *Concrete International*, Vol. 8, No. 10, pp. 17-19.

Erlemann, G.G., 1997, "Reinforcing Bar Specifications – 1911 through 1968," *Engineered Concrete Structures*, Portland Cement Association, Skokie, Illinois.

Erlemann, G.G., 1999, "Steel reinforcing bar specifications in old structures," *Concrete International*, Vol. 21, No. 4, pp. 49-50.

EOTA, 2013, *High Strength Reinforcing System SAS 670*, European Technical Approval ETA-13/0840, European Organization for Technical Approvals, Vienna, Austria.

Fei, J., and Darwin, D., 1999, *Fatigue of High Relative Rib Area Reinforcing Bars*, SM Report No. 54, University of Kansas Center for Research, Inc., Lawrence, Kansas.

FEMA, 2000, *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*, FEMA 350, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 2009, *Quantification of Building Seismic Performance Factors*, FEMA P-695, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

Frosch, R.J., 1999, "Another look at cracking and crack control in reinforced concrete," *ACI Structural Journal*, Vol. 96, No. 3, pp. 437-442.

Frosch, R.J., 2001, "Flexural crack control in reinforced concrete," *Design and Construction Practices to Mitigate Cracking*, ACI Special Publication 204, American Concrete Institute, Farmington Hills, Michigan.

Frosch, R.J., 2002, "Modeling and control of side face beam cracking," *ACI Structural Journal*, Vol. 99, No. 3, pp. 376-385.

Gamble, W.L., 2011, *Loads, Codes, and Fire Endurance*, ACI Special Publication 279, American Concrete Institute, Farmington Hills, Michigan.

Garay-Moran, J.D., and Lubell, A.S., 2008, "Behavior of concrete deep beams with high strength reinforcement," *Proceedings*, ASCE Structures Congress, Reston, Virginia.

Gaston, J.R., and Hognestad, E., 1962, "High strength bars as concrete reinforcement, Part 3 – Tests of full-scale roof girder," *Journal of the PCA Research and Development Laboratories*, Vol. 4, No. 2, pp. 10-23.

Gergely, P., and Lutz, L.A., 1968, "Maximum crack width in reinforced concrete flexural members," *Causes, Mechanism, and Control of Cracking in Concrete*, ACI Special Publication 20, American Concrete Institute, Detroit, Michigan.

Hage, S.E., 1974, *The Second-Order Analysis of Reinforced Concrete Frames*, M.S. Thesis, University of Alberta, Edmonton, Canada.

Han, S.W., and Jee, N.Y., 2005, "Seismic behaviors of columns in ordinary and intermediate moment resisting concrete frames," *Engineering Structures*, Vol. 27, No. 6, pp. 951-962.

Helgason, T., Hanson, J.M., Somes, N.F., Corley, W.G., and Hognestad, E., 1976, *Fatigue Strength of High-Yield Reinforcing Bars*, National Cooperative Highway Research Program Report 164, Transportation Research Board of the National Academies, Washington, D.C.

Hognestad, E., 1961, "High strength bars as concrete reinforcement, Part 1 – Introduction to a series of experimental reports," *Journal of the PCA Research and Development Laboratories*, Vol. 3, No. 3, pp. 23-29.

Hognestad, E., 1962, "High strength bars as concrete reinforcement, Part 2 – Control of flexural cracking," *Journal of the PCA Research and Development Laboratories*, Vol. 4, No. 1, pp. 46-63.

Hopkins, D., and Poole, R., 2005, *Grade 500E Reinforcing Steel, Tests on Micro-Alloy and Quenched and Tempered Samples Available in New Zealand*, Department of Building and Housing, Wellington, New Zealand.

Hwang, H.H.M., and Hsu, H.M., 1993, "Seismic LRFD Criteria for RC Moment-Resisting Frame Buildings," *Journal of Structural Engineering,* Vol. 119, No. 6, pp. 1807-1824.

Hwang, H.H.M., and Hsu, H.M., 1994, "A study of reliability-based seismic design criteria," *Structural Safety & Reliability*, ISBN 90 5410 357 4, Balkema, Rotterdam.

ICC, 2012, *International Building Code*, International Code Council, Washington, D.C.

ICC-ES, 2011, Evaluation Subject: SAS Stressteel Grade 97 Thread Bar Steel Reinforcing Bars and Couplers, ESR-1163, International Code Council Evaluation Services, Whittier, California.

Ishikawa, Y., Kimura, H., Takatsu, H., and Ousalem, H., 2008, "Ultimate deformation of R/C columns using high-strength concrete and high-strength steel bars under earthquake loading," *Proceedings*, 8th International Symposium on Utilization of High-Strength and High-Performance Concrete, S1-5-4, Tokyo, Japan.

Jhamb, I.C., and MacGregor, J. G., 1974, "Effect of Surface Characteristics On Fatigue Strength of Reinforcing Steel," *Abeles Symposium on Fatigue of Concrete,* ACI Special Publication 41, American Concrete Institute, Detroit, Michigan, pp. 139-167.

Kaar, P.H., 1966, "High strength bars as concrete reinforcement, Part 8 – Similitude in flexural cracking of T-beam flanges," *Journal of the PCA Research and Development Laboratories*, Vol. 8, No. 2, pp. 2-12.

Kaar, P.H., and Hognestad, E., 1965, "High strength bars as concrete reinforcement, Part 7 – Control of cracking in T-beam flanges," *Journal of the PCA Research and Development Laboratories*, Vol. 7, No. 1, pp. 42-53.

Kaar, P.H., and Mattock, A.H., 1963, “High strength bars as concrete reinforcement, Part 4 – Control of cracking,” *Journal of the PCA Research and Development Laboratories*, Vol. 5, No. 1, pp. 15-38.

Kabeyasawa, T., and Hiraishi, H., 1998, “Tests and analyses of high-strength reinforced concrete shear walls in Japan,” *High-Strength Concrete in Seismic Regions*, ACI Special Publication 176, American Concrete Institute, Farmington Hills, Michigan.

Kimura, H., Sugano, S., Nagashima, T., and Ichikawa, A., 1993, “Seismic loading tests of reinforced concrete beams using high strength concrete and high strength steel bars,” *Proceedings*, 3rd International Symposium on Utilization of High-Strength Concrete, pp. 377-384, Tokyo, Japan.

Kimura, H., and Ishikawa, Y., 2008, “Seismic performance of high-strength reinforced concrete slender walls subjected to high axial loading,” *Proceedings*, 8th International Symposium on Utilization of High-Strength and High-Performance Concrete, pp. 945-950, Tokyo, Japan.

Korinda, K., 1972, “Discussion No. 3 – Cracking and crack control,” *Proceedings*, International Conference on Planning and Design of Tall Buildings, Lehigh University, Bethlehem, Pennsylvania.

Kudder, R.J., and Gustafson, D.P., 1983, “Bend tests of Grade 60 reinforcing bars,” *ACI Journal Proceedings*, Vol. 80, No. 3, pp. 202-209.

Larkins, T., 2004, *Shear Design of Reinforced Concrete Beams: Investigation of an Alternative Approach*, M.S. Thesis, Purdue University, West Lafayette, Indiana.

Lin, C.M., Restrepo, J.I., and Park, R., 2000, *Seismic Behaviour and Design of Reinforced Concrete Interior Beam Column Joints*, Research Report 2000-1, Department of Civil Engineering, University of Canterbury, New Zealand.

Lowes, L.N., Lehman, D.E., Birely, A.C., Kuchma, D.A., Marley, K.P., and Hart, C.R., 2012, “Earthquake response of slender planar concrete walls with modern detailing,” *Engineering Structures*, Vol. 43, pp. 31-47.

LSTC, 2013, *LS-DYNA*, Livermore Software Technology Corporation, Livermore, California, http://www.lstc.com/products/ls-dyna.

Lubell, A., Sherwood, T., Bentz, E., and Collins, M.P., 2004, "Safe shear design of large, wide beams," *ACI Concrete International*, Vol. 26, No. 1, pp. 66-78.

Luna, B.N., 2014 (in preparation), *Seismic Response of Low Aspect Ratio Reinforced Concrete Shear Walls for Buildings and Safety-Related Nuclear Structures*, Ph.D. Dissertation, University at Buffalo, Buffalo, New York.

MacGregor, J.G., and Hage, S.E., 1977, "Stability analysis and design of concrete frames," *Journal of the Structural Division*, ASCE, Vol. 103, No. 10, pp. 1953-1970.

Marques, J., and Jirsa, J., 1975, "A study of hooked bar anchorages in beam-column joints," *ACI Structural Journal*, Vol. 72, No. 5, pp. 198-209.

Mast, R.F., 1968, "Auxiliary reinforcement in concrete connections," *Journal of the Structural Division*, ASCE, Vol. 94, No. 6, pp. 1485-1504.

Mast, R.F., 1992, "Unified design provisions for reinforced and prestressed concrete flexural and compression members," *ACI Structural Journal*, Vol. 89, No. 2, pp. 185-199

Mattock, A.H., and Hawkins, N.M., 1972, "Shear transfer in reinforced concrete – recent research," *PCI Journal*, Vol. 17, No. 2, pp. 55-75.

Mattock, A.H., Yamazaki, J., and Kattula, B.T., 1971, "Comparative study of prestressed concrete beams, with and without bond," *ACI Journal Proceedings*, Vol. 68, No. 2, pp. 116-125.

Meinheit, D.F., and Felder, A.L., 2014, *Vintage Steel Reinforcement in Concrete Structures*, Concrete Reinforcing Steel Institute, Schaumburg, Illinois.

Moyer, M.J., and Kowalsky, M.J., 2003, "Influence of tension strain on buckling of reinforcement in concrete columns," *ACI Structural Journal*, Vol. 100, No. 1, pp. 75-85.

Muguruma, H., and Watanabe, F., 1990, "Ductility improvement of high-strength concrete columns with lateral confinement," *High-Strength Concrete: Second International Symposium*, ACI Special Publication 121, American Concrete Institute, Detroit, Michigan.

Munikrishna, A., 2008, *Shear Behavior of Concrete Beams Reinforced with High Performance Steel Shear Reinforcement*, M.S. Thesis, North Carolina State University, Raleigh, North Carolina.

NACU, 1910, *Standard Building Regulations for the Use of Reinforced Concrete*, Standard 4, National Association of Cement Users, Philadelphia, Pennsylvania.

Nishiyama, M., 2009, "Mechanical properties of concrete and reinforcement state-of-the-art report on HSC and HSS in Japan," *Journal of Advanced Concrete Technology*, Vol. 7, No. 2, pp. 157-182.

NIST, 2010a, *Seismic Design of Cast-in-Place Concrete Diaphragms, Chords, and Collectors–A Guide for Practicing Engineers*, NIST GCR 10-917-4 Report, prepared by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology, Gaithersburg, Maryland.

NIST, 2010, *Evaluation of the FEMA P-695 Methodology for Quantification of Building Seismic Performance Factors*, GCR 10-917-8, prepared by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium for Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology, Gaithersburg, Maryland.

NIST, 2012, *Tentative Framework for Development of Advanced Seismic Design Criteria for New Buildings*, GCR 12-917-20, prepared by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium for Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology, Gaithersburg, Maryland.

NIST, 2014, *Use of High-Strength Reinforcement in Earthquake-Resistant Concrete Structures*, GCR 14-917-30, prepared by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium for Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology, Gaithersburg, Maryland.

Okamoto, M., Sato, T., Tanimura, Y., and Kuroiwa, T., 2004, "Experimental study on seismic performance verification method for RC members using high-strength materials," *Journal of Advanced Concrete Technology*, Vol. 2, No. 2, pp. 223-231.

OpenSees, 2011, *Open System for Earthquake Engineering Simulation: OpenSees*, University of California, Berkeley, http://opensees.berkeley.edu.

Orangun, C.O., Jirsa, J.O., and Breen, J.E., 1975, *The Strength of Anchored Bars: A Reevaluation of Test Data on Development Length and Splices*, Research Report No. 154-3F, Center for Highway Research, University of Texas, Austin, Texas.

Ospina, C.E., 2005, "Alternative for concentric punching capacity evaluation of reinforced concrete two-way slabs," *Concrete International*, Vol. 27, No. 9, pp. 53-57.

Otani, S., 1991, "AIJ proposal of ultimate strength design requirements for RC buildings with emphasis on beam-column joints," *Design of Beam-Column Joints for Seismic Resistance*, ACI Special Publication 123, American Concrete Institute, Detroit, Michigan.

Ou, Y., Kurniawan, D., and Handika, N., 2012, "Shear behavior of reinforced concrete columns with high strength steel and concrete under low axial load," *Proceedings*, ACI Fall Convention, Toronto, Ontario, Canada.

Pan, A., and Moehle, J.P., 1989, "Lateral displacement ductility of reinforced concrete flat plates," *Structural Journal*, Vol. 86, No. 3, pp. 250-258.

Panahshahi, N., and Lu, C., 1997, "Inelastic Response of RC Intermediate and Special Moment Resisting Frames," *Proceedings*, ASCE Structures Congress, Portland, Oregon.

Paulson, C., and Hanson, J.M., 1991, *Final Report: Fatigue Strength of Welded and Mechanical Splices in Reinforcing Steel*, National Cooperative Highway Research Program, Project 10-35, Transportation Research Board, Washington, D.C.

Paulson, C., Graham, S.K., and Rautenberg, J.M., 2013, *Determination of Yield Strength for Nonprestressed Steel Reinforcement*, prepared by Wiss, Janney, Elstner Associates, Inc., for the Charles Pankow Foundation, RGA 04-13, WJE No. 2013.4171, Pasadena, California.

Pfister, J.F., 1964, "Influence of ties on the behavior of reinforced concrete columns," *ACI Journal Proceedings*, Vol. 61, No. 5, pp. 521-538.

Pfister, J.F., and Hognestad, E., 1964, "High strength bars as concrete reinforcement, Part 6 – Fatigue tests," *Journal of the PCA Research and Development Laboratories*," Vol. 6, No. 1, pp. 65-84.

Pfister, J.F., and Mattock, A.H., 1963, "High strength bars as concrete reinforcement, Part 5 – Lapped splices in concentrically loaded columns," *Journal of the PCA Research and Development Laboratories*, Vol. 5, No. 2, pp. 27-40.

Pfund, S.J., 2012, *Cyclic Response of Concrete Beams Reinforced with ASTM A1035 Grade-120 Steel Bars*, M.S. Thesis, The Pennsylvania State University, University Park, Pennsylvania.

Price, K.R., Fields, D., and Lowes, L.N., 2014, *The Impact of High-Strength Reinforcing Steel on Current Design Practice*, Research Grant Agreement #01-13, Charles Pankow Foundation, Vancouver, Washington.

Rautenberg, J.M., 2011, *Drift Capacity of Concrete Columns Reinforced with High Strength Steel*, Ph.D. Thesis, Purdue University, West Lafayette, Indiana.

Reineck, K.H., Kuchma, D.A., Kim, K.S., and Marx, S., 2003, "Shear database for reinforced concrete members without shear reinforcement," *ACI Structural Journal*, Vol. 100, No. 2, pp. 240-249.

Restrepo, J.I., Seible, F., Stephan, B., and Schoettler, M.J., 2006, "Seismic testing of bridge columns incorporating high-performance materials," *ACI Structural Journal*, Vol. 103, No. 4, pp. 496-504.

Restrepo-Posada, J.I., 1992, *Seismic Behaviour of Connections Between Precast Concrete Elements*, Ph.D. Thesis, University of Canterbury, Christchurch, New Zealand.

Richard, M.J., Albano, L.D., Kelly, D.J., and Liel, A., 2010, "Case study on the seismic performance of reinforced-concrete intermediate-moment frames using ACI design provisions," *Proceedings*, NASCC Steel Conference/ASCE Structures Congress, Orlando, Florida.

Richart, F.E., 1933, "Reinforced concrete column investigation," *ACI Journal Proceedings*, Vol. 29, No. 2, pp. 275-284.

Richart, F.E., and Brown, R.L., 1934, *An Investigation of Reinforced Concrete Columns*, Engineering Experiment Station Bulletin No. 267, University of Illinois, Urbana, Illinois.

Rodriguez, M., Botero, J., and Villa, J., 1999, "Cyclic stress-strain behavior of reinforcing steel including effect of buckling," *Journal of Structural Engineering*, Vol. 125, No. 6, pp. 605–612.

Schlaich, J., Schafer, K., and Jennewein, M., 1987, "Toward a consistent design of structural concrete," *PCI Journal*, Vol. 32, No. 3, pp. 74-150.

Seliem, H.M., Hosny, A., Rizkalla, S., Zia, P., Briggs, M., Miller, S., Darwin, D., Browning, J., Glass, G.M., Hoyt, K., Donnelly, K., and

Jirsa, J.O., 2009, "Bond characteristics of high-strength ASTM A1035 steel reinforcing bars," *ACI Structural Journal*, Vol. 106, No. 4, pp. 530-539.

Shahrooz, B.M., Miller, R.A., Harris, K.A., and Russell, H.G., 2011, *Design of Concrete Structures Using High-Strength Steel Reinforcement*, National Cooperative Highway Research Program (NCHRP) Report 679, Transportation Research Board of the National Academies, Washington, D.C.

Sheth, A., 2003, "Use of intermediate RC moment frames in moderate seismic zones," *The Indian Concrete Journal,* Vol. 77, No. 11, pp. 1431-1435.

Sozen, M.A., Siess, C.P., 1963 "Investigation of multiple panel reinforced concrete floor slabs," *ACI Journal Proceedings*, Vol. 60, No. 8, pp. 999-1028.

Stecich, J.P., Hanson, J.M., and Rice, P.F., 1984, "Bending and straightening of Grade 60 reinforcing bars," *Concrete International*, Vol. 6, No. 8, pp. 14-23.

Sugano, S., Nagashima, T., Kimura, H., Tamura, A., and Ichikawa, A., 1990, "Experimental studies on seismic behavior of reinforced concrete members of high-strength concrete," *High-Strength Concrete: Second International Symposium*, ACI Special Publication 121, American Concrete Institute, Detroit, Michigan.

Sumpter, M.S., Rizkalla, S.H., and Zia, P., 2009, "Behavior of high-performance steel as shear reinforcement for concrete beams," *ACI Structural Journal*, Vol. 106, No. 2, pp. 171-177.

Tanaka, H., 1990, *Effect of Lateral Confining Reinforcement on the Ductile Behaviour of Reinforced Concrete Columns*, Ph.D. Thesis, University of Canterbury, Christchurch, New Zealand.

Tavallali, H., 2011, *Cyclic Response of Concrete Beams Reinforced with Ultrahigh Strength Steel*, Ph.D. Thesis, Pennsylvania State University, University Park, Pennsylvania.

Thomas, K., and Sozen, M.A., 1965, *A Study of the Inelastic Rotation Mechanism of Reinforced Concrete Connections*, Civil Engineering Studies, Structural Research Series No. 301, University of Illinois, Urbana, Illinois.

TNO, 2013, *Displacement Analyzer*, DIANA, TNO DIANA BV, Delft, The Netherlands, http://tnodiana.com/content/DIANA.

Todeschini, C.E., Bianchini, A.C., and Kesler, C.E., 1964, “Behavior of concrete columns reinforced with high strength steels,” *ACI Journal Proceedings*, Vol. 61, No. 6, pp. 701-716.

Topcu, I.B., and Karakurt, C., 2008, “Properties of reinforced concrete steel rebars exposed to high temperatures,” *Research Letters in Materials Science*, Vol. 2008, Article ID 814137.

Tretiakova, K., 2013, *Cyclic Response of Concrete Columns Reinforced with SAS 670 Grade-97 Steel Bars*, M.S. Thesis, Pennsylvania State University, University Park, Pennsylvania.

Tureyen, A.K., and Frosch, R.J., 2002, “Shear tests of FRP-reinforced concrete beams without stirrups,” *ACI Structural Journal*, Vol. 99, No. 4, pp. 427-434.

Tureyen, A.K., and Frosch, R.J., 2003, “Concrete shear strength: Another perspective,” *ACI Structural Journal*, Vol. 100, No. 5, pp. 609-615.

Valluvan, R., 1993, *Issues Involved In Seismic Retrofit of Reinforced Concrete Frames Using Infilled Walls*, Ph.D. Thesis, University of Texas, Austin, Texas.

Wang, Y., and Restrepo, J.I., 2001, “Investigation of concentrically loaded reinforced concrete columns confined with glass fiber-reinforced polymer jackets,” *ACI Structural Journal*, Vol. 98, No. 3, pp. 377-385.

Wood, S.L., 1989, “Minimum tensile reinforcement requirements in walls,” *ACI Structural Journal*, Vol. 86, No. 5, pp. 582-591.

Zheng, H., and Abel, A., 1999, “Fatigue Properties of Reinforcing Steel Produced by TEMPCORE Process,” *Journal of Materials in Civil Engineering,* ASCE, Vol. 11, No. 2, pp. 158-165.

Zhu, S., and Jirsa, J.O., 1983, *A Study of Bond Deterioration in Reinforced Concrete Beam-Column Joints*, PMFSEL Report No. 83-1, Department of Civil Engineering, University of Texas, Austin, Texas.

Ziehl, P.H., Cloyd, J.E., and Kreger, M.E., 1998, *Evaluation of Minimum Longitudinal Reinforcement Requirements for Reinforced Concrete Columns*, Research Report 1473-S, Center for Transportation Research, University of Texas, Austin, Texas.

Ziehl, P.H., Cloyd, J.E., and Kreger, M.E., 2004, “Investigation of minimum longitudinal reinforcement requirements for concrete columns using present-day construction materials," *ACI Structural Journal*, Vol. 101, No. 2, pp. 165-175.

Zuo, J., and Darwin, D., 2000, "Splice strength of conventional and high relative rib area bars in normal and high-strength concrete," *ACI Structural Journal*, Vol. 97, No. 4, pp. 630-641.

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Applied Technology Council Projects and Report Information

One of the primary purposes of the Applied Technology Council is to develop engineering applications and resources that translate and summarize useful information for practicing building and bridge design professionals. This includes the development of guidelines and manuals, as well as the development of research recommendations for specific areas determined by the profession. ATC is not a code development organization, although ATC project reports often serve as resource documents for the development of codes, standards and specifications.

Applied Technology Council conducts projects that meet the following criteria:

1. The primary audience or benefactor is the design practitioner in structural engineering.
2. A cross section or consensus of engineering opinion is required to be obtained and presented by a neutral source.
3. The project fosters the advancement of structural engineering practice.

Funding for projects is obtained from government agencies and tax-deductible contributions from the private sector. Brief descriptions of completed ATC projects and reports are provided below.

ATC-1: This project resulted in five papers published as part of *Building Practices for Disaster Mitigation, Building Science Series 46*, proceedings of a workshop sponsored by the National Science Foundation (NSF) and the National Bureau of Standards (NBS). Available through the National Technical Information Service (NTIS), 5285 Port Royal Road, Springfield, VA 22151, as NTIS report No. COM-73-50188.

ATC-2: The report, *An Evaluation of a Response Spectrum Approach to Seismic Design of Buildings*, was funded by NSF and NBS and was conducted as part of the Cooperative Federal Program in Building Practices for Disaster Mitigation. Available through ATC. (Published 1974, 270 Pages)

ATC-3: The report, *Tentative Provisions for the Development of Seismic Regulations for Buildings* (ATC-3-06), was funded by NSF and NBS. The tentative provisions in this report served as the basis for the seismic provisions of the 1988 and subsequent issues of the *Uniform Building Code* and the *NEHRP Recommended Provisions for the Development of Seismic Regulation for New Building and Other Structures*. The second printing contains proposed amendments prepared by a joint committee of the Building Seismic Safety Council (BSSC) and the NBS. Available through ATC. (Published 1978, amended 1982, 505 pages plus proposed amendments)

ATC-3-2: The project, "Comparative Test Designs of Buildings Using ATC-3-06 Tentative Provisions", was funded by NSF. It consisted of a study to develop and plan a program for making comparative test designs of the ATC-3-06 Tentative Provisions. The project report was intended for use by the Building Seismic Safety Council in its refinement of the ATC-3-06 Tentative Provisions.

ATC-3-4: The report, *Redesign of Three Multistory Buildings: A Comparison Using ATC-3-06 and 1982 Uniform Building Code Design Provisions*, was published under a grant from NSF. Available through ATC. (Published 1984, 112 pages)

ATC-3-5: The project, "Assistance for First Phase of ATC-3-06 Trial Design Program Being Conducted by the Building Seismic Safety Council," was funded by the Building Seismic Safety Council to obtain assistance in conducting the first phase of its program to develop trial designs for buildings in Los Angeles, Seattle, Phoenix, and Memphis.

ATC-3-6: The project, "Assistance for Second Phase of ATC-3-06 Trial Design Program Being Conducted by the Building Seismic Safety Council," was funded by the Building Seismic Safety Council to obtain assistance in conducting the second phase of its program to develop trial designs for buildings in New York, Chicago, St. Louis, Charleston, and Fort Worth.

ATC-4: The report, *A Methodology for Seismic Design and Construction of Single-Family Dwelling*s, was published under a contract with the Department of Housing and Urban Development (HUD). Available through ATC. (Published 1976, 576 pages)

ATC-4-1: The report, *The Home Builders Guide for Earthquake Design*, was published under a contract with HUD. Available through ATC. (Published 1980, 57 pages)

ATC-5: The report, *Guidelines for Seismic Design and Construction of Single-Story Masonry Dwellings in Seismic Zone 2*, was developed under a contract with HUD. Available through ATC. (Published 1986, 38 pages)

ATC-6: The report, *Seismic Design Guidelines for Highway Bridges*, was published under a contract with the Federal Highway Administration (FHWA). Available through ATC. (Published 1981, 210 pages)

ATC-6-1: The report, *Proceedings of a Workshop on Earthquake Resistance of Highway Bridges*, was published under a grant from NSF. Available through ATC. (Published 1979, 625 pages)

ATC-6-2: The report, *Seismic Retrofitting Guidelines for Highway Bridges*, was published under a contract with FHWA. Available through ATC. (Published 1983, 220 pages)

ATC-7: The report, *Guidelines for the Design of Horizontal Wood Diaphragms*, was published under a grant from NSF. Available through ATC. (Published 1981, 190 pages)

ATC-7-1: The report, *Proceedings of a Workshop on Design of Horizontal Wood Diaphragms*, was published under a grant from NSF. Available through ATC. (Published 1980, 302 pages)

ATC-8: The report, *Proceedings of a Workshop on the Design of Prefabricated Concrete Buildings for Earthquake Loads*, was funded by NSF. Available through ATC. (Published 1981, 400 pages)

ATC-9: The report, *An Evaluation of the Imperial County Services Building Earthquake Response and Associated Damage*, was published under a grant from NSF. Available through ATC. (Published 1984, 231 pages)

ATC-10: The report, *An Investigation of the Correlation Between Earthquake Ground Motion and Building Performance*, was funded by the U.S. Geological Survey (USGS). Available through ATC. (Published 1982, 114 pages)

ATC-10-1: The report, *Critical Aspects of Earthquake Ground Motion and Building Damage Potential*, was co-funded by the USGS and the NSF. Available through ATC. (Published 1984, 259 pages)

ATC-11: The report, *Seismic Resistance of Reinforced Concrete Shear Walls and Frame Joints: Implications of Recent Research for Design Engineers*, was published under a grant from NSF. Available through ATC. (Published 1983, 184 pages)

ATC-12: The report, *Comparison of United States and New Zealand Seismic Design Practices for Highway Bridges*, was published under a grant from NSF. Available through ATC. (Published 1982, 270 pages)

ATC-12-1: The report, *Proceedings of Second Joint U.S.-New Zealand Workshop on Seismic Resistance of Highway Bridges*, was published under a grant from NSF. Available through ATC. (Published 1986, 272 pages)

ATC-13: The report, *Earthquake Damage Evaluation Data for California*, was developed under a contract with the Federal Emergency Management Agency (FEMA). It presents expert-opinion earthquake damage and loss estimates for industrial, commercial, residential, utility and transportation facilities in California. Included are damage probability matrices for 78 classes of structures and estimates of time required to restore damaged facilities to pre-earthquake usability. Available through ATC. (Published 1985, 492 pages)

ATC-13-1: The report, *Commentary on the Use of ATC-13 Earthquake Damage Evaluation Data for Probable Maximum Loss Studies of California Buildings*, was developed with funding from the ATC Endowment Fund. It provides guidance for using ATC-13 expert-opinion data for probable maximum loss (PML) studies of California buildings. Included are discussions of the limitations on the use of the ATC-13 expert-opinion data, and appendices containing information not included in the original ATC-13 report, such as model building type descriptions, beta damage distribution parameters for ATC-13 model building types, and PML values for ATC-13 model building types. Available through ATC. (Published 2002, 66 pages)

ATC-14: The report, *Evaluating the Seismic Resistance of Existing Buildings*, was developed under a grant from the NSF. It describes a methodology for performing preliminary and detailed seismic evaluations of buildings. A precursor to the eventual ASCE 31 Standard, *Seismic Evaluation of Existing Buildings*, it contains useful background information including a state-of-practice review; seismic loading criteria; data collection procedures; a detailed description of the building classification system; preliminary and detailed analysis procedures; and example case studies, including nonstructural considerations. Available through ATC. (Published 1987, 370 pages)

ATC-15: The report, *Comparison of Seismic Design Practices in the United States and Japan*, was published under a grant from NSF. Available through ATC. (Published 1984, 317 pages)

ATC-15-1: The report, *Proceedings of Second U.S.-Japan Workshop on Improvement of Building Seismic Design and Construction Practices*, was published under a grant from NSF. It includes state-of-the-practice papers and case studies of actual building designs and information on regulatory, contractual, and licensing issues. Available through ATC. (Published 1987, 412 pages)

ATC-15-2: The report, *Proceedings of Third U.S.-Japan Workshop on Improvement of Building Structural Design and Construction Practices*, was published jointly by ATC and the Japan Structural Consultants Association. It includes state-of-the-practice papers on steel braced frame and reinforced concrete buildings, base isolation and passive energy dissipation devices, and comparisons between U.S. and Japanese design practice. Available through ATC. (Published 1989, 358 pages)

ATC-15-3: The report, *Proceedings of Fourth U.S.-Japan Workshop on Improvement of Building Structural Design and Construction Practices*, was published jointly by ATC and the Japan Structural Consultants Association. It includes papers on postearthquake building damage assessment; acceptable earthquake damage; repair and retrofit of earthquake-damaged buildings; base-isolated buildings, Architectural Institute of Japan recommendations for design; active damping systems; and wind-resistant design. Available through ATC. (Published 1992, 484 pages)

ATC-15-4: The report, *Proceedings of Fifth U.S.-Japan Workshop on Improvement of Building Structural Design and Construction Practices*, was published jointly by ATC and the Japan Structural Consultants Association. It includes papers on performance goals and acceptable damage; seismic design procedures and case studies; seismic evaluation, repair and upgrade; construction influences on design; isolation and passive energy dissipation; design of irregular structures; and quality control for design and construction. Available through ATC. (Published 1994, 360 pages)

ATC-16: The FEMA 90 report, *An Action Plan for Reducing Earthquake Hazards of Existing Buildings*, was funded by FEMA and was conducted by a joint venture of ATC, the Building Seismic Safety Council and the Earthquake Engineering Research Institute. Available through FEMA. (Published 1985, 75 pages)

ATC-17: The report, *Proceedings of a Seminar and Workshop on Base Isolation and Passive Energy Dissipation*, was published under a grant from NSF. It includes papers describing case studies in the United States, applications and developments worldwide, recent innovations in technology development, and structural and ground motion issues in base-isolation and passive energy-dissipation. Also included is a proposed 5-year research agenda. Available through ATC. (Published 1986, 478 pages)

ATC-17-1: The report, *Proceedings of a Seminar on Seismic Isolation, Passive Energy Dissipation and Active Control,* was published under a grant from NCEER and NSF. Available through ATC. (Published 1993, 841 pages in two volumes)

ATC-18: The report, *Seismic Design Criteria for Bridges and Other Highway Structures: Current and Future***,** was developed under a grant from NCEER and FHWA. Available through ATC. (Published, 1997, 151 pages)

ATC-18-1: The report, *Impact Assessment of Selected MCEER Highway Project Research on the Seismic Design of Highway Structures***,** was developed under a contract with the Multidisciplinary Center for Earthquake Engineering Research (MCEER, formerly

NCEER) and FHWA. Available through ATC. (Published, 1999, 136 pages)

ATC-19: The report, *Structural Response Modification Factors* was funded by NSF and NCEER. Available through ATC. (Published 1995, 70 pages)

ATC-20: The report, *Procedures for Postearthquake Safety Evaluation of Buildings*, was developed under a contract with the California Office of Emergency Services (OES), California Office of Statewide Health Planning and Development (OSHPD) and FEMA. It provides procedures and guidelines for inspecting buildings that have been damaged in an earthquake, and making decisions regarding their continued use and occupancy. Written for volunteer structural engineers and building inspectors, it includes rapid and detailed evaluation procedures for posting buildings as "inspected" (apparently safe, green placard), "limited entry" (yellow) or "unsafe" (red). Available through ATC (Published 1989, 152 pages)

ATC-20-1: The report, *Field Manual: Postearthquake Safety Evaluation of Buildings, Second Edition*, was funded by Applied Technology Council. A companion to the ATC-20 report, the *Field Manual* summarizes postearthquake safety evaluation procedures in a concise format designed for ease of use in the field. Available through ATC. (Published 2004, 143 pages)

ATC-20-1 Bhutan: The report, Bhutan Field Manual: Postearthquake Safety Evaluation of Buildings, was developed in partnership with GeoHazards International and the Royal Government of Bhutan's Department of Engineering Services and Department of Disaster Management with funding from the ATC Endowment Fund and the World Bank's Global Facility for Disaster Reduction in Recovery. The Bhutan Field Manual is an adaptation of the postearthquake safety evaluation procedures described in ATC-20 to account for Bhutan's vernacular buildings, as well as Bhutan's cultural and governmental context. Available through ATC. (Published 2014, 246 pages)

ATC-20-2: The report, *Addendum to the ATC-20 Postearthquake Building Safety Procedures* was published under a grant from the NSF and funded by the USGS. It provides updated assessment forms, placards, and evaluation procedures based on application and use in five earthquake events that occurred after the initial release of the ATC-20 report. Available through ATC. (Published 1995, 94 pages)

ATC-20-3: The report, *Case Studies in Rapid Postearthquake Safety Evaluation of Buildings,* was funded by ATC and R.P. Gallagher Associates. Containing over 50 case studies using the ATC-20 Rapid Evaluation procedure, the report is intended for use as a training and reference manual describing how buildings are inspected and evaluated. Illustrated with photos and completed safety assessment forms and placards. Available through ATC. (Published 1996, 295 pages)

ATC-20-T: The *Postearthquake Safety Evaluation of Buildings Training CD* was developed in cooperation with FEMA. The 4½-hour training seminar includes photographs, schematic drawings, and textual information. Available through ATC. (Published 2002, 230 PowerPoint slides with Speakers Notes)

ATC-21: The FEMA 154 report, *Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook*, *Second Edition*, was developed under a contract with FEMA. It describes a rapid visual screening procedure for identifying buildings that might pose serious risk of loss of life and injury in the event of a damaging earthquake. Available through ATC and FEMA. (Published 2002, 161 pages)

ATC-21-1: The FEMA 155 report, *Rapid Visual Screening of Buildings for Potential Seismic Hazards: Supporting Documentation*, *Second Edition*, was developed under a contract with FEMA. It provides the technical basis for the updated rapid visual screening procedure. Available through ATC and FEMA. (Published 2002, 117 pages)

ATC-21-2: The report, *Earthquake Damaged Buildings: An Overview of Heavy Debris and Victim Extrication*, was developed under a contract with FEMA. (Published 1988, 95 pages)

ATC-21-T: The report, *Rapid Visual Screening of Buildings for Potential Seismic Hazards Training Manual, Second Edition,* was developed under a contract with FEMA. Training materials include120 slides in PowerPoint format and companion narrative coordinated with the presentation. Available through ATC. (Published 2004, 148 pages and PowerPoint presentation on companion CD)

ATC-22: The report, *A Handbook for Seismic Evaluation of Existing Buildings (Preliminary)*, was developed under a contract with FEMA in 1989. Based on the information originally developed in ATC-14, this report was revised by BSSC and published as the FEMA 178 report, *NEHRP Handbook for the Seismic Evaluation of Existing Buildings* in 1992, revised by ASCE and published as the FEMA 310 report, *Handbook for the Seismic Evaluation of Buildings – a Prestandard* in 1998. Currently available through the American Society of Civil Engineers as the ASCE 31 Standard, *Seismic Evaluation of Existing Buildings*.

ATC-22-1: The report, *Seismic Evaluation of Existing Buildings: Supporting Documentation*, was developed under a contract with FEMA. (Published 1989, 160 pages)

ATC-23A: The report, *General Acute Care Hospital Earthquake Survivability Inventory for California, Part A: Survey Description, Summary of Results, Data Analysis and Interpretation*, was developed under a contract with the Office of Statewide Health Planning and Development (OSHPD), State of California. Available through ATC. (Published 1991, 58 pages)

ATC-23B: The report, *General Acute Care Hospital Earthquake Survivability Inventory for California, Part B: Raw Data*, was developed under a contract with the Office of Statewide Health Planning and Development (OSHPD), State of California. Available through ATC. (Published 1991, 377 pages)

ATC-24: The report, *Guidelines for Seismic Testing of Components of Steel Structures*, was jointly funded by the American Iron and Steel Institute (AISI), American Institute of Steel Construction (AISC), National Center for Earthquake Engineering Research (NCEER), and NSF. Available through ATC. (Published 1992, 57 pages)

ATC-25: The report, *Seismic Vulnerability and Impact of Disruption of Lifelines in the Conterminous States*, was developed under a contract with FEMA. Available through ATC. (Published 1991, 440 pages)

ATC-25-1: The report, *A Model Methodology for Assessment of Seismic Vulnerability and Impact of Disruption of Water Supply Systems*, was developed under a contract with FEMA. Available through ATC. (Published 1992, 147 pages)

ATC-26: This project, "U.S. Postal Service National Seismic Program," was funded under a contract with the U.S. Postal Service (USPS), and resulted in the following interim documents:

ATC-26 Report, *Cost Projections for the U. S. Postal Service Seismic Program* (Completed 1990)

ATC-26-1 Report, *United States Postal Service Procedures for Seismic Evaluation of Existing Buildings (Interim)* (Completed 1991)

ATC-26-2 Report, *Procedures for Post-disaster Safety Evaluation of Postal Service Facilities (Interim)*. Available through ATC. (Published 1991, 221 pages)

ATC-26-3 Report, *Field Manual: Post-earthquake Safety Evaluation of Postal Buildings (Interim)*. Available through ATC. (Published 1992, 133 pages)

ATC-26-3A Report, *Field Manual: Post Flood and Wind Storm Safety Evaluation of Postal Buildings (Interim)*. Available through ATC. (Published 1992, 114 pages)

ATC-26-4 Report, *United States Postal Service Procedures for Building Seismic Rehabilitation (Interim)* (Completed 1992)

ATC-26-5 Report, *United States Postal Service Guidelines for Building and Site Selection in Seismic Areas (Interim)* (Completed 1992)

ATC-28: The report, *Development of Recommended Guidelines for Seismic Strengthening of Existing Buildings, Phase I: Issues Identification and Resolution*, was developed under a contract with FEMA. Available through ATC. (Published 1992, 150 pages)

ATC-29: The report, *Proceedings of a Seminar and Workshop on Seismic Design and Performance of Equipment and Nonstructural Elements in Buildings and Industrial Structures*, was developed under a grant from NCEER and NSF. It includes papers describing current practice, codes and regulations; earthquake performance; analytical and experimental investigations; development of new seismic qualification methods; and research, practice, and code development needs for nonstructural elements and systems. Available through ATC. (Published 1992, 470 pages)

ATC-29-1: The report, *Proceedings of a Seminar on Seismic Design, Retrofit, and Performance of Nonstructural Components*, was developed under a grant from NCEER and NSF. It includes papers on observed performance in recent earthquakes; seismic design codes, standards, and procedures for commercial and institutional buildings; design issues relating to industrial and hazardous material facilities; and seismic evaluation and rehabilitation of components in conventional and essential facilities. Available through ATC. (Published 1998, 518 pages)

ATC-29-2: The report, *Proceedings of Seminar on Seismic Design, Performance, and Retrofit of Nonstructural Components in Critical Facilities*, was developed under a grant from MCEER (formerly NCEER) and NSF. It includes papers on seismic design, performance, and retrofit of nonstructural components in critical facilities including current practices and emerging codes; seismic design and retrofit; risk and performance evaluation; system qualification and testing; and advanced technologies. Available through ATC. (Published 2003, 574 pages)

ATC-30: The report, *Proceedings of Workshop for Utilization of Research on Engineering and Socioeconomic Aspects of 1985 Chile and Mexico Earthquakes*, was developed under a grant from the NSF. Available through ATC. (Published 1991, 113 pages)

ATC-31: The report, *Evaluation of the Performance of Seismically Retrofitted Buildings*, was developed under a contract with the National Institute of Standards and Technology (NIST, formerly NBS) and funded by the USGS. Available through ATC. (Published 1992, 75 pages)

ATC-32: The report, *Improved Seismic Design Criteria for California Bridges: Provisional Recommendations,* was funded by the California Department of Transportation (Caltrans). Available through ATC. (Published 1996, 215 pages)

ATC-32-1: The report, *Improved Seismic Design Criteria for California Bridges: Resource Document,* was funded by Caltrans. Available through ATC. (Published 1996, 365 pages; also available on CD)

ATC-33: The project, funded under a contract with the Building Seismic Safety Council, was initiated by FEMA to develop nationally applicable, state-of-the-art guidance for performance-based seismic rehabilitation of buildings. Work resulted in the publication of:

FEMA 273, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (Published 1997, 440 pages). Revised by ASCE and published as the FEMA 356 report, *Prestandard and Commentary for the Seismic Rehabilitation of Buildings* in 2000. Currently available through the American Society of Civil Engineers as the ASCE 41 Standard, *Seismic Rehabilitation of Existing Buildings*.

FEMA 274, *NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings*. Available through ATC and FEMA. (Published 1997, 492 pages)

FEMA 276, *Example Applications of the NEHRP Guidelines for the Seismic Rehabilitation of Buildings*. Available through ATC and FEMA. (Published 1997, 295 pages)

ATC-34: The report, *A Critical Review of Current Approaches to Earthquake Resistant Design***,** was developed under a grant from NCEER and NSF. Available through ATC. (Published, 1995, 94 pages)

ATC-35: The report, *Enhancing the Transfer of U.S. Geological Survey Research Results into Engineering Practice* was developed under a cooperative agreement with the USGS. Available through ATC. (Published 1994, 120 pages)

ATC-35-1: The report, *Proceedings of Seminar on New Developments in Earthquake Ground Motion Estimation and Implications for Engineering Design Practice*, was developed under a cooperative agreement with USGS. It includes papers describing state-of-the-art information on regional earthquake risk; new techniques for estimating strong ground motions as a function of earthquake source, travel path, and site parameters; and new developments applicable to geotechnical engineering. Available through ATC. (Published 1994, 478 pages)

ATC-35-2: The report, *Proceedings: National Earthquake Ground Motion Mapping Workshop*, was developed under a cooperative agreement with USGS. It includes papers on ground motion parameters; reference site conditions; probabilistic versus deterministic basis; and the treatment of uncertainty in seismic source characterization and ground motion attenuation. Available through ATC. (Published 1997, 154 pages)

ATC-35-3: The report, *Proceedings: Workshop on Improved Characterization of Strong Ground Shaking for Seismic Design,* was developed under a cooperative agreement with USGS. It includes papers on identifying needs and developing improved representations of earthquake ground motion for use in seismic design practice and building codes. Available through ATC. (Published 1999, 75 pages)

ATC-37: The report, *Review of Seismic Research Results on Existing Buildings***,** was developed in conjunction with the Structural Engineers Association of California (SEAOC) and California Universities for Research in Earthquake Engineering (CUREe) under a contract with the California Seismic Safety Commission (SSC). Available through the Seismic Safety Commission as Report SSC 94-03. (Published, 1994, 492 pages)

ATC-38: The report, *Database on the Performance of Structures near Strong-Motion Recordings: 1994 Northridge, California, Earthquake*, was developed with funding from the USGS, the Southern California Earthquake Center (SCEC), OES, and the Institute for Business and Home Safety (IBHS). Available through ATC. (Published 2000, 260 pages, with CD containing complete database).

ATC-40: The report, *Seismic Evaluation and Retrofit of Concrete Buildings***,** was developed under a contract with the California Seismic Safety Commission. It provides guidance on performance objectives, hazard characterization, identification of deficiencies, retrofit strategies, nonlinear static analysis procedures, modeling rules, foundation effects, and response limits for seismic evaluation and retrofit of concrete buildings. Available through ATC. (Published, 1996, 612 pages in two volumes)

ATC-41 (SAC Joint Venture, Phase 1): The project, "Program to Reduce the Earthquake Hazards of Steel Moment-Resisting Frame Structures, Phase 1," was funded by FEMA and OES and conducted by a Joint Venture partnership of SEAOC, ATC, and CUREe. Under Phase 1 the following documents were prepared:

SAC-94-01, *Proceedings of the Invitational Workshop on Steel Seismic Issues, Los Angeles, September 1994*. Available through ATC. (Published 1994, 155 pages)

SAC-95-01, *Steel Moment-Frame Connection Advisory No. 3*. Available through ATC. (Published 1995, 310 pages)

SAC-95-02, *Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment-Frame Structures* (FEMA 267 report) (Published 1995, 215 pages; superseded by FEMA 350 to 353)

SAC-95-03, *Characterization of Ground Motions During the Northridge Earthquake of January 17, 1994*. Available through ATC. (Published 1995, 179 pages)

SAC-95-04, *Analytical and Field Investigations of Buildings Affected by the Northridge Earthquake of January 17, 1994*. Available through ATC. (Published 1995, 900 pages in two volumes)

SAC-95-05, *Parametric Analytical Investigations of Ground Motion and Structural Response, Northridge Earthquake of January 17, 1994*. Available through ATC. (Published 1995, 274 pages)

SAC-95-06, *Surveys and Assessment of Damage to Buildings Affected by the Northridge Earthquake of January 17, 1994*. Available through ATC. (Published 1995, 315 pages)

SAC-95-07, *Case Studies of Steel Moment Frame Building Performance in the Northridge Earthquake of January 17, 1994* (Published 1995, 260 pages, Available through ATC)

SAC-95-08, *Experimental Investigations of Materials, Weldments and Nondestructive Examination Techniques*. Available through ATC. (Published 1995, 144 pages)

SAC-95-09, *Background Reports: Metallurgy, Fracture Mechanics, Welding, Moment Connections and Frame systems, Behavior* (FEMA 288 report). Available through ATC and FEMA. (Published 1995, 361 pages)

SAC-96-01, *Experimental Investigations of Beam-Column Subassemblages, Part 1 and 2*. Available through ATC. (Published 1996, 924 pages, in two volumes)

SAC-96-02, *Connection Test Summaries* (FEMA 289 report). Available through ATC and FEMA. (Published 1996, 144 pages)

ATC-41-1 (SAC Joint Venture, Phase 2): The project, "Program to Reduce the Earthquake Hazards of Steel Moment-Resisting Frame Structures, Phase 2," was funded by FEMA and conducted by a Joint Venture partnership of

SEAOC, ATC, and CUREe. Under Phase 2 the following documents were prepared:

SAC-96-03, *Interim Guidelines Advisory No. 1 Supplement to FEMA 267 Interim Guidelines* (FEMA 267A report) (Published 1997, 100 pages; superseded by FEMA 350 to 353)

SAC-99-01, *Interim Guidelines Advisory No. 2 Supplement to FEMA 267 Interim Guidelines* (FEMA 267B report, superseding FEMA 267A). (Published 1999, 150 pages; superseded by FEMA 350 to 353)

FEMA 350, *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*. Available through ATC and FEMA. (Published 2000, 190 pages)

FEMA 351, *Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings*. Available through ATC and FEMA. (Published 2000, 210 pages)

FEMA 352, *Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings*. Available through ATC and FEMA. (Published 2000, 180 pages)

FEMA 353, *Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications*. Available through ATC and FEMA. (Published 2000, 180 pages)

FEMA 354, *A Policy Guide to Steel Moment-Frame Construction*. Available through ATC and FEMA. (Published 2000, 27 pages)

FEMA 355A, *State of the Art Report on Base Materials and Fracture*. Available through ATC and FEMA. (Published 2000, 107 pages; in print and on CD).

FEMA 355B, *State of the Art Report on Welding and Inspection*. Available through ATC and FEMA. (Published 2000, 185 pages; in print and on CD).

FEMA 355C, *State of the Art Report on Systems Performance of Steel Moment Frames Subject to Earthquake Ground Shaking*. Available through ATC and FEMA. (Published 2000, 322 pages; in print and on CD).

FEMA 355D, *State of the Art Report on Connection Performance*. Available through ATC and FEMA. (Published 2000, 292 pages; in print and on CD).

FEMA 355E, *State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquakes*. Available through ATC and FEMA. (Published 2000, 190 pages; in print and on CD).

FEMA 355F, *State of the Art Report on Performance Prediction and Evaluation of Steel Moment-Frame Structures*. Available through ATC and FEMA. (Published 2000, 347 pages; in print and on CD).

ATC-43: The reports, *Evaluation of Earthquake-Damaged Concrete and Masonry Wall Buildings, Basic Procedures Manual* (FEMA 306), *Evaluation of Earthquake-Damaged Concrete and Masonry Wall Buildings, Technical Resources* (FEMA 307), and *The Repair of Earthquake Damaged Concrete and Masonry Wall Buildings* (FEMA 308), were developed for FEMA under a contract with the Partnership for Response and Recovery, a Joint Venture of Dewberry & Davis and Woodward-Clyde. Available through ATC and FEMA. (Published, 1998 in print and on CD; *Basic Procedures Manual*, 270 pages; *Technical Resources*, 271 pages; *Repair Manual*, 81 pages)

ATC-44: The report, *Hurricane Fran, North Carolina, September 5, 1996: Reconnaissance Report*, was funded by the Applied Technology Council. Available through ATC. (Published 1997, 36 pages)

ATC-45: The report, *Field Manual*, *Safety Evaluation of Buildings After Wind Storms and Floods*, was developed with funding from the ATC Endowment Fund and the Institute for Business and Home Safety (IBHS). It provides rapid and detailed evaluation procedures for inspecting buildings that have been damaged in wind storms and floods, and making decisions regarding their continued use and occupancy. Presented in a concise format designed for ease of use in the field, it is intended for use by volunteer structural engineers and building inspectors in posting buildings as “inspected” (apparently safe, green placard), “restricted use” (yellow) or “unsafe” (red). Available through ATC. (Published 2004, 132 pages)

ATC-48 (ATC/SEAOC Joint Venture Training Curriculum): The training curriculum, *Built to Resist Earthquakes, The Path to Quality Seismic Design and Construction for Architects, Engineers, and Inspectors*, was developed under a contract with the California Seismic Safety Commission and prepared by a Joint Venture partnership between ATC and SEAOC. Available through ATC. (Published 1999, 314 pages)

ATC-49: The 2-volume report, *Recommended LRFD Guidelines for the Seismic Design of Highway Bridges; Part I: Specifications* and *Part II: Commentary and Appendices*, were developed under the ATC/MCEER Joint Venture partnership with funding from the Federal Highway Administration. Available through ATC. (Published 2003, *Part I*, 164 pages and *Part II*, 294 pages)

ATC-49-1: The document, *Liquefaction Study Report, Recommended LRFD Guidelines for the Seismic Design of Highway Bridges,* was developed under the ATC/MCEER Joint Venture partnership with funding from the Federal Highway Administration. Available through ATC. (Published 2003, 208 pages)

ATC-49-2: The report, *Design Examples, Recommended LRFD Guidelines for the Seismic Design of Highway Bridges*, was developed under the ATC/MCEER Joint Venture partnership with funding from the Federal Highway Administration. Available through ATC. (Published 2003, 316 pages)

ATC-51: The report, *U.S.-Italy Collaborative Recommendations for Improved Seismic Safety of Hospitals in Italy*, was developed under a contract with Servizio Sismico Nazionale of Italy (Italian National Seismic Survey). Available through ATC. (Published 2000, 154 pages)

ATC-51-1: The report, *Recommended U.S.-Italy Collaborative Procedures for Earthquake Emergency Response Planning for Hospitals in Italy*, was developed under a contract with Servizio Sismico Nazionale of Italy (Italian National Seismic Survey, NSS). Available in English and Italian through ATC. (Published 2002, 120 pages)

ATC-51-2: The report, *Recommended U.S.-Italy Collaborative Guidelines for Bracing and Anchoring Nonstructural Components in Italian Hospitals*, was developed under a contract with the Department of Civil Protection, Italy. Available in English and Italian through ATC. (Published 2003, 164 pages)

ATC-52: The project, "Development of a Community Action Plan for Seismic Safety (CAPSS), City and County of San Francisco", was conducted under a contract with the San Francisco Department of Building Inspection. The following reports were prepared:

ATC-52-1, *Here Today—Here Tomorrow: The Road to Earthquake Resilience in San Francisco: Potential Earthquake Impacts*. Available through ATC. (Published 2010, 78 pages)

ATC-52-1A, *Here Today—Here Tomorrow: The Road to Earthquake Resilience in San Francisco: Potential Earthquake Impacts Technical Documentation*. Available through ATC. (Published 2010, 160 pages)

ATC-52-2, *Here Today—Here Tomorrow: The Road to Earthquake Resilience in San Francisco: A Community Action Plan for Seismic Safety*. Available through ATC. (Published 2010, 92 pages)

ATC-52-3, *Here Today—Here Tomorrow: The Road to Earthquake Resilience in San Francisco: Earthquake Safety for Soft-Story Buildings*. Available through ATC. (Published 2009, 60 pages)

ATC-52-3A, *Here Today—Here Tomorrow: The Road to Earthquake Resilience in San Francisco: Earthquake Safety for Soft-Story Buildings Documentation Appendices*. Available through ATC. (Published 2009, 206 pages)

ATC-52-4, *Here Today—Here Tomorrow: The Road to Earthquake Resilience in San Francisco: Post-Earthquake Repair and Retrofit Requirements*. Available through ATC. (Published 2010, 130 pages)

ATC-53: The report, *Assessment of the NIST 12-Million-Pound (53 MN) Large-Scale Testing Facility*, was developed under a contract with NIST. Available through ATC. (Published 2000, 44 pages)

ATC-54: The report, *Guidelines for Using Strong-Motion Data and ShakeMaps in Postearthquake Response*, was developed under a contract with the California Geological Survey. Available through ATC. (Published 2005, 222 pages)

ATC-55: The FEMA 440 report, *Improvement of Nonlinear Static Seismic Analysis Procedures*, was developed under a contract with FEMA. Available through ATC and FEMA. (Published 2005, 152 pages)

ATC-56: The report, FEMA 389, *Primer for Design Professionals: Communicating with Owners and Managers of New Buildings on Earthquake Risk*, was developed under a contract with FEMA. Available through ATC and FEMA. (Published 2004, 194 pages)

ATC-56-1: The report, FEMA 427, *Primer for Design of Commercial Buildings to Mitigate Terrorist Attacks – Providing Protection to People and Buildings*, was developed under a contract with FEMA. Available through ATC and FEMA. (Published 2003, 106 pages)

ATC-57: The report, *The Missing Piece: Improving Seismic Design and Construction Practices*, was developed under a contract with NIST. It provides a framework for eliminating the technology transfer gap that has emerged within the National Earthquake Hazards Reduction Program (NEHRP) that limits the adaptation of basic research knowledge into practice. Available through ATC. (Published 2003, 102 pages)

ATC-58: The ATC-58/ATC-58-1/ATC-58-2 series of projects, "Development of Next-Generation Performance-Based Seismic Design Guidelines for New and Existing Buildings," was a multi-year, multi-phase effort funded by FEMA that resulted in the publication of the following:

ATC-58-1, *Proceedings of a FEMA-Sponsored Workshop on Communicating Earthquake Risk*. Available through ATC. (Published 2002, 87 pages).

ATC-58-2, *Preliminary Evaluation of Methods for Defining Performance*. Available through ATC. (Published 2003, 99 pages).

ATC-58-3, *Proceedings of a FEMA-Sponsored Workshop on Performance-Based Design*. Available through ATC. (Published 2003, 146 pages).

ATC-58-4, *Proceedings of a FEMA-Sponsored Workshop on Communicating Seismic Performance Metrics in Design Decision-Making*. Available through ATC. (Published 2014, 73 pages).

FEMA 445, *Next-Generation Performance-Based Seismic Design Guidelines, Program Plan for New and Existing Buildings*. Available through ATC and FEMA. (Published 2006, 131 pages).

FEMA 461, *Interim Testing Protocols for Determining the Seismic Performance Characteristics of Structural and Nonstructural Components*. Available through ATC and FEMA. (Published 2007, 113 pages).

FEMA P-58-1, *Seismic Performance Assessment of Buildings, Volume 1 – Methodology*. Available through ATC and FEMA. (Published 2012, 319 pages).

FEMA P-58-2, *Seismic Performance Assessment of Buildings, Volume 2 – Implementation Guide*. Available through ATC and FEMA. (Published 2012, 365 pages).

FEMA P-58-3, *Seismic Performance Assessment of Buildings, Volume 3 – Supporting Electronic Materials and Background Documentation*. Available through ATC and FEMA. (Published 2012, on CD).

FEMA P-58-4, *Seismic Performance Assessment of Buildings, Volume 4 – Methodology for Assessing Environmental Impacts*. Available through ATC and FEMA. (Published 2012, 120 pages)

ATC-60: The 2-volume report, *SEAW Commentary on Wind Code Provisions, Volume 1* and *Volume 2 - Example Problems,* was developed by the Structural Engineers Association of Washington (SEAW) in cooperation with ATC. Available through ATC. (Published 2004; *Volume 1*, 238 pages; *Volume 2*, 245 pages)

ATC-61: The 2-volume report, *Natural Hazard Mitigation Saves: An Independent Study to Assess the Future Savings from Mitigation Activities, Volume 1 – Findings, Conclusions, and Recommendations*, and *Volume 2 – Study Documentation,* was prepared for the Multihazard Mitigation Council (MMC) of the National Institute of Building Sciences, with funding provided by FEMA. Available through ATC and the MMC. (Published 2005; *Volume 1*, 11 pages; *Volume 2*, 366 pages)

ATC-62: The report, FEMA P-440A, *Effects of Strength and Stiffness Degradation on Seismic Response*, was developed under a contract with FEMA. Developed as a supplement to the FEMA 440 report, it provides additional guidance on modeling of nonlinear degrading response. Available through ATC and FEMA. (Published 2009, 310 pages)

ATC-63: The report, FEMA P-695, *Quantification of Building Seismic Performance*

Factors, was developed under a contract with FEMA. It describes a methodology for establishing seismic performance factors (R , Ω_0, and C_d) that involves the development of detailed system design information and probabilistic assessment of collapse risk. Available through ATC and FEMA. (Published 2009, 420 pages)

ATC-63-1: The report, FEMA P-795, *Quantification of Building Seismic Performance Factors: Component Equivalency Methodology*, was developed under a contract with FEMA. Available through ATC and FEMA. (Published 2011, 264 pages)

ATC-64: The reports, *Guidelines for Design of Structures for Vertical Evacuation from Tsunamis* (FEMA P-646), and *Vertical Evacuation from Tsunamis: A Guide for Community Officials* (FEMA P-646A), were developed under a contract with FEMA. Available through ATC and FEMA. (*Design Guidelines*, Published 2008, 174 pages; *Guide for Community Officials*, Published 2009, 62 pages)

ATC-65: The FEMA P-455 report, *Handbook for Rapid Visual Screening of Buildings to Evaluate Terrorism Risks*, was developed under a contract with FEMA. Available through ATC and FEMA. (Published 2009, 174 pages)

ATC-66: The FEMA P-774 report, *Unreinforced Masonry Buildings and Earthquakes, Developing Successful Risk Reduction Programs*, was developed under a contract with FEMA. Available through ATC and FEMA. (Published 2009, 194 pages)

ATC-67: The *Rapid Observation of Vulnerability and Estimation of Risk* (ROVER) smartphone application was developed in collaboration with specialists form SPA Risk LLC, and Instrumental Software Technologies Inc. under a contract with FEMA. It is intended for use by building professionals (engineers, architects, firefighters, building officials, and others) to do pre-earthquake screening and post-earthquake safety evaluation of buildings in an electronic format. Available through ATC and FEMA. (Published 2014, online and on CD)

ATC-68: The FEMA P-420 report, *Engineering Guideline for Incremental Seismic Rehabilitation*, was developed under a contract with FEMA. Available through ATC and FEMA. (Published 2009, 94 pages)

ATC-69: The report, *Reducing the Risks of Nonstructural Earthquake Damage, State-of-the-Art and Practice Report*, was developed under a contract with FEMA. Available through ATC and FEMA. (Published 2008, 144 pages)

ATC-69-1: The electronic document, FEMA E-74, *Reducing the Risks of Nonstructural Earthquake Damage, A Practical Guide*, was developed under a contract with FEMA. Available through ATC and FEMA. (Published 2011, 750 pages)

ATC-70: The report, NIST Technical Note 1476, *Performance of Physical Structures in Hurricane Katrina and Hurricane Rita: A Reconnaissance Report*, was developed under a contract with NIST. Available through NIST. (Published 2006, 222 pages)

ATC-71: The reports, *Workshop on Meeting the Challenges of Existing Buildings, Part 1 Workshop Proceedings; Part 2: Status Report on Seismic Evaluation and Rehabilitation of Existing Buildings;* and *Part 3: Action Plan for the FEMA Existing Buildings Program*, were developed under a contract with FEMA. Available through ATC and FEMA. (*Part 1*, Published 2008, 142 pages; *Part 2*, Published 2009, 140 pages; *Part 3*, Published 2009, 118 pages)

ATC-71-1: The FEMA P-807 report, *Seismic Evaluation and Retrofit of Multi-Unit Wood-Frame Buildings with Weak First Stories*, was developed under a contract with FEMA. Available through ATC and FEMA. (Published 2012, 230 pages, including the *Weak Story Tool* on CD)

ATC-71-2: The report, *Proceedings: Workshop on a Rating System for the Earthquake Performance of Buildings*, was developed under a contract with FEMA. Available through ATC. (Published 2011, 102 pages)

ATC-71-4/ATC-71-5/ATC-71-6: The FEMA P-154 report, *Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook, Third Edition*, and the FEMA P-155 Report, *Rapid Visual Screening of Buildings for Potential Seismic Hazards: Supporting Documentation, Third Edition*, were developed under a series of contracts with FEMA. Available through ATC and FEMA. (Published, 2014; *Handbook*, 388 pages; *Supporting Documentation*, 206 pages)

ATC-72: The report, *Proceedings of Workshop on Tall Building Seismic Design and Analysis Issues* (ATC-72) was prepared for the Building Seismic Safety Council of the National Institute of Building Sciences, with funding provided by

FEMA. The report, *Modeling and Acceptance Criteria for Seismic Design and Analysis of Tall Buildings* (PEER/ATC-72-1) was prepared for the Pacific Earthquake Engineering Research Center. Available through ATC and PEER. (*Proceedings*, Published 2007, 84 pages; *Modeling and Acceptance Criteria*, Published 2010, 242 pages)

ATC-73: The report, *NEHRP Workshop on Meeting the Challenges of Existing Buildings, Prioritized Research for Reducing the Seismic Hazards of Existing Buildings,* was developed under a grant from NSF. Available through ATC. (Published 2007, 22 pages)

ATC-74: The report, *Collaborative Recommended Requirements for Automatic Natural Gas Shutoff Valves in Italy,* was funded by the Department of Civil Protection, Italy. Available through ATC. (Published 2007, 76 pages)

ATC-75: The report, *Improvements to BIM Structural Software Interoperability*, was developed under a contract with the Charles Pankow Foundation. Available through ATC and CPF. (Published 2013, 155 pages)

ATC-76-1/ATC-76-4: The report, *Evaluation of the FEMA P-695 Methodology for the Quantification of Building Seismic Performance Factors,* was developed under a contract with NIST and prepared by a Joint Venture partnership between ATC and CUREE. Available through ATC, CUREE, and NIST as GCR 10-917-8. (Published 2010, 240 pages)

ATC-76-2: The report, *Program Plan for the Development of Seismic Design Guidelines for Port Container, Wharf, and Cargo Systems,* was developed under a contract with NIST and prepared by a Joint Venture partnership between ATC and CUREE. Available through ATC, CUREE, and NIST as GCR 12-917-19. (Published 2012, 134 pages)

ATC-76-3: The reports, *NEHRP Technical Brief No. 1, Seismic Design of Reinforced Concrete Special Moment Frames: A Guide for Practicing Engineers* and *NEHRP Technical Brief No. 2, Seismic Design of Steel Special Moment Frames: A Guide for Practicing Engineers,* were developed under a contract with NIST and prepared by a Joint Venture partnership between ATC and CUREE. Available through ATC, CUREE, and NIST (*Technical Brief No. 1*, Report GCR 08-917-1. Published 2008, 32 pages; *Technical Brief No. 2*, Report GCR 09-917-3, Published 2009, 38 pages)

ATC-76-5: The report, *Program Plan for the Development of Collapse Assessment and Mitigation Strategies for Existing Reinforced Concrete Buildings,* was developed under a contract with NIST and prepared by a Joint Venture partnership between ATC and CUREE. Available through ATC, CUREE, and NIST as GCR 10-917-7. (Published 2010, 80 pages)

ATC-76-6: The report, *Applicability of Nonlinear Multiple-Degree-of-Freedom Modeling for Design,* was developed under a contract with NIST and prepared by a Joint Venture partnership between ATC and CUREE. Available through ATC, CUREE, and NIST as GCR 10-917-9. (Published 2010, 196 pages plus CD)

ATC-76-7: The report, *NEHRP Technical Brief No. 3, Seismic Design of Cast-in-Place Concrete Diaphragms, Chords, and Collectors: A Guide for Practicing Engineers,* was developed under a contract with NIST and prepared by a Joint Venture partnership between ATC and CUREE. Available through ATC, CUREE, and NIST as GCR 10-917-4. (Published 2010, 30 pages)

ATC-76-8: The report, *NEHRP Technical Brief No. 4, Nonlinear Structural Analysis for Seismic Design: A Guide for Practicing Engineers*, was developed under a contract with NIST and prepared by a Joint Venture partnership between ATC and CUREE. Available through ATC, CUREE, and NIST as GCR 10-917-5. (Published 2010, 32 pages)

ATC-76-9: The project, "Performance of Two Full Scale Reinforced Concrete Subassemblage Tests," was funded by NIST to perform tests in support of an internal research program to develop computer models for predicting the collapse potential of reinforced concrete structures. Work was conducted under a Joint Venture partnership between ATC and CUREE.

ATC-78: The report, *Identification and Mitigation of Seismically Hazardous Older Concrete Buildings: Interim Methodology Evaluation* (ATC-78), and its successor report, *Evaluation of the Methodology to Select and Prioritize Collapse Indicators in Older Concrete Buildings* (ATC-78-1), were developed under a contract with FEMA. ATC-78-1 is currently available through ATC. (Published 2012, 153 pages)

ATC-79: The FEMA P-646 report, *Guidelines for Design of Structures for Vertical Evacuation from Tsunamis, Second Edition*, was developed under a contract with FEMA. The original version of the report was developed under the ATC-64 Project. Available through ATC and FEMA. (Published 2012, 194 pages)

ATC-82: The report, *Selecting and Scaling Earthquake Ground Motions for Performing Response-History Analyses*, was developed under a contract with NIST and prepared by a Joint Venture partnership between ATC and CUREE. Available through ATC, CUREE, and NIST as GCR 11-917-5. (Published 2011, 234 pages)

ATC-83: The report, *Soil-Structure Interaction for Building Structures*, was developed under a contract with NIST and prepared by a Joint Venture partnership between ATC and CUREE. Available through ATC, CUREE, and NIST as GCR 12-917-21. (Published 2012, 292 pages)

ATC-84: The report, *Tentative Framework for Development of Advanced Seismic Design Criteria for New Buildings*, was developed under a contract with NIST and prepared by a Joint Venture partnership between ATC and CUREE. Available through ATC, CUREE, and NIST as GCR 12-917-20. (Published 2012, 302 pages)

ATC-85: The project, "Assessment of ASCE 41 First Generation Performance-Based Seismic Design Methods for new Buildings in High-Seismic Regions: Phases I-III," was funded by NIST to obtain technical assistance on the initiation of an internal research project benchmarking ASCE 41 performance-based seismic design procedures as applied to new buildings designed in accordance with ASCE 7. Work was conducted under a Joint Venture partnership between ATC and CUREE.

ATC-86: The report, FEMA P-58-4, *Seismic Performance Assessment of Buildings, Volume 4 – Methodology for Assessing Environmental Impacts*, was developed under a contract with FEMA in support of the ATC-58 Project. Available through ATC and FEMA. (Published 2012, 120 pages)

ATC-87: The report, *NEHRP Technical Brief No. 5, Seismic Design of Composite Steel Deck and Concrete-filled Diaphragms: A Guide for Practicing Engineers*, was developed under a contract with NIST and prepared by a Joint Venture partnership between ATC and CUREE. Available through ATC, CUREE, and NIST as GCR 11-917-4. (Published 2011, 34 pages)

ATC-88: The report, *NEHRP Technical Brief No. 6, Seismic Design of Cast-in-Place Concrete Special Structural Walls and Coupling Beams: A Guide for Practicing Engineers*, was developed under a contract with NIST and prepared by a Joint Venture partnership between ATC and CUREE. Available through ATC, CUREE, and NIST as GCR 11-917-11. (Published 2011, 38 pages)

ATC-89: The report, *Cost Analyses and Benefit Studies for Earthquake-Resistant Construction in Memphis, Tennessee*, was developed under a contract with NIST and prepared by a Joint Venture partnership between ATC and CUREE. Available through ATC, CUREE, and NIST as GCR 14-917-26. (Published 2014, 227 pages)

ATC-90: The report, *Research Plan for the Study of Seismic Behavior and Design of Deep, Slender Wide Flange Structural Steel Beam-Column Members*, was developed under a contract with NIST and prepared by a Joint Venture partnership between ATC and CUREE. Available through ATC, CUREE, and NIST as GCR 11-917-13. (Published 2011, 148 pages)

ATC-91: The project, "Assessment of Nonlinear Seismic Analysis of Structures Based on Modal Superposition," was funded by NIST to obtain technical support for an internal research program investigating the use of a new approach to nonlinear analysis. Work was conducted under a Joint Venture partnership between ATC and CUREE.

ATC-92: The report, *Comparison of U.S. and Chilean Building Code Requirements and Seismic Design Practice 1985–2010*, was developed under a contract with NIST and prepared by a Joint Venture partnership between ATC and CUREE. Available through ATC, CUREE, and NIST as GCR 12-917-18. (Published 2012, 110 pages)

ATC-93: The project, "Ground Motion and Building Performance Data From the 2010 Chile Earthquake," was funded by NIST to develop a prototypical web-based repository for post-event data in support of the NIST Disaster and Failure Events Database initiative. Work was conducted under a Joint Venture partnership between ATC and CUREE.

ATC-94: The report, *Recommendations for Seismic Design of Reinforced Concrete Wall Buildings Based on Studies of the 2010 Maule,* *Chile Earthquake*, was developed under a contract with NIST and prepared by a Joint Venture partnership between ATC and CUREE. Available through ATC, CUREE, and NIST as GCR 14-917-25. (Published 2014, 321 pages)

ATC-95: The report, *Review of Past Performance and Further Development of Modeling Techniques for Collapse Assessment of Existing Reinforced Concrete Buildings*, was developed under a contract with NIST and prepared by a Joint Venture partnership between ATC and CUREE. Available through ATC, CUREE, and NIST as GCR 14-917-28. (Published 2014, 201 pages)

ATC-96: The report, *Nonlinear Analysis Research and Development Program for Performance-Based Seismic Engineering*, was developed under a contract with NIST and prepared by a Joint Venture partnership between ATC and CUREE. Available through ATC, CUREE, and NIST as GCR 14-917-27. (Published 2014, 147 pages)

ATC-97: The report, *NEHRP Technical Brief No. 7, Seismic Design of Reinforced Concrete Mat Foundations: A Guide for Practicing Engineers*, was developed under a contract with NIST and prepared by a Joint Venture partnership between ATC and CUREE. Available through ATC, CUREE, and NIST as GCR 12-917-22. (Published 2012, 34 pages)

ATC-98: The report, *Use of High-Strength Reinforcement in Earthquake-Resistant Concrete Structures*, was developed under a contract with NIST and prepared by a Joint Venture partnership between ATC and CUREE. Available through ATC, CUREE, and NIST as GCR 14-917-30. (Published 2014, 231 pages)

ATC-99: The project, "Methodology to Assess and Verify the Seismic Capacity of Low-Rise Buildings," was funded by FEMA to study an alternative seismic design approach for low-rise construction in the United States.

ATC-100: The report, *Measurement Science R&D Roadmap for Windstorm and Coastal Inundation Impact Reduction*, was developed under a contract with NIST and prepared by a Joint Venture partnership between ATC and CUREE. Available through ATC, CUREE, and NIST as GCR 14-973-13. (Published 2014, 130 pages)

ATC-101: The report, A *Framework to Update the Plan to Coordinate NEHRP Post-Earthquake Investigations*, was developed under a contract with NIST and prepared by a Joint Venture partnership between ATC and CUREE. Available through ATC, CUREE, and NIST as GCR 14-917-29. (Published 2014, 103 pages)

ATC-102: The report, *Earthquake-Resilient Lifelines: NEHRP Research, Development and Implementation Roadmap*, was developed under a contract with NIST and prepared by a Joint Venture partnership between ATC and CUREE. Available through ATC, CUREE, and NIST as GCR 14-917-33. (Published 2014, 163 pages)

ATC-103: The report, *NEHRP Technical Brief No. 8, Seismic Design of Steel Special Concentrically Braced Frame Systems: A Guide for Practicing Engineers*, was developed under a contract with NIST and prepared by a Joint Venture partnership between ATC and CUREE. Available through ATC, CUREE, and NIST as GCR 13-917-24. (Published 2013, 36 pages)

ATC-104: The project, "Assessment of the Performance of Slender Reinforced Concrete Walls under Significant Lateral Loads," was funded by NIST to obtain technical support for an internal research project investigating the behavior of reinforced concrete shear walls. Work was conducted under a Joint Venture partnership between ATC and CUREE.

ATC-105: The project, "Development of Annual Report for National Earthquake Hazards Reduction Program Covering Fiscal Year 2012," was funded by NIST to obtain assistance in the development of the NEHRP Annual Report in 2013. Work was conducted under a Joint Venture partnership between ATC and CUREE.

ATC-106: The project, "Seismic Behavior and Design of Deep, Slender Wide-Flange Structural Steel Beam-Column Members: Phase 2 Experimental Evaluation," was funded by NIST to perform testing in support of an internal research program investigating the behavior of steel beam-column members. Work was conducted under a Joint Venture partnership between ATC and CUREE.

ATC-107: The project, "Wind Speed Mapping," was funded by NIST to obtain technical assistance in the development of revised wind speed maps incorporating NIST non-tropical wind analysis at different return periods. Work was conducted under a Joint Venture partnership between ATC and CUREE.

ATC-108: The project, "Assessment of ASCE 41 First Generation Performance-Based Seismic

Design Methods for new Buildings in High-Seismic Regions" was funded by NIST to obtain technical assistance on the completion of an internal research project benchmarking ASCE 41 performance-based seismic design procedures as applied to new buildings designed in accordance with ASCE 7. Work was conducted under a Joint Venture partnership between ATC and CUREE.

ATC-110: The report, *Plan for Development of a Prestandard for Evaluation and Retrofit of Wood Light-Frame Dwellings*, was developed under a contract with the California Earthquake Authority (CEA) in collaboration with FEMA. Available through ATC and CEA. (Published 2014, 85 pages)

ATC-111: The report, *NEHRP Technical Brief No. 9, Seismic Design of Special Reinforced Masonry Shear Walls: A Guide for Practicing Engineers*, was developed under a contract with NIST, and prepared in collaboration with CUREE. Available through ATC and NIST as GCR 14-917-31. (Published 2014, 42 pages)

ATC-112: The report, *NEHRP Technical Brief No. 10, Seismic Design of Wood Light-Frame Structural Diaphragm Systems: A Guide for Practicing Engineers*, was developed under a contract with NIST, and prepared in collaboration with CUREE. Available through ATC and NIST as GCR 14-917-32. (Published 2014, 47 pages)

ATC-113: The project, "Development of Annual Report for National Earthquake Hazards Reduction Program Covering Fiscal Year 2013," was funded by NIST to obtain assistance in the development of the NEHRP Annual Report in 2014.

ATC-118: The FEMA P-1019 report, *Emergency Power Systems for Critical Facilities: A Best Practices Approach to Improving Reliability*, was developed under a contract with FEMA. Available through ATC and FEMA. (Published 2014, 170 pages)

ATC-R-1: The report, *Cyclic Testing of Narrow Plywood Shear Walls*, was developed with funding from the ATC Endowment Fund. Available through ATC (Published 1995, 64 pages)

ATC Design Guide 1: The report, *Minimizing Floor Vibration*, was developed with funding from the ATC Endowment Fund. Available through ATC. (Published, 1999, 64 pages)

ATC Design Guide 2: The report, *Basic Wind Engineering for Low-Rise Buildings*, was developed with funding from the ATC Endowment Fund. Available through ATC. (Published, 2009, 114 pages)

ATC TechBrief 1: The ATC TechBrief 1, *Liquefaction Maps*, was developed under a contract with the United States Geological Survey. Available through ATC. (Published 1996, 12 pages)

ATC TechBrief 2: The ATC TechBrief 2, *Earthquake Aftershocks − Entering Damaged Buildings*, was developed under a contract with the United States Geological Survey. Available through ATC. (Published 1996, 12 pages)

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