TALL BUILDING CONCRETE SHEAR WALL DESIGN USING HIGH STRENGTH GRADE 80 LONGITUDINAL REINFORCING IN SAN FRANCISCO

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Abstract

The design of concrete shear wall building longitudinal reinforcement for seismic performance has traditionally been limited to a yield strength of 60 ksi. A substantial amount of testing and analysis effort is currently being made in the structural engineering community to incorporate the use of high strength reinforcement into building systems. ASTM A706 Grade 80 reinforcement, with special additional requirements, is being used as concrete shear wall longitudinal reinforcement for the 1500 Mission Street development in San Francisco. This paper addresses the reinforcement analysis and design parameters used for these two performance based designed buildings and the specific reinforcement testing requirements specified to confirm material performance. The road map presented will provide guidance for future similar building designs.

Keywords

High Strength Reinforcement; ASTM A706 Grade 80; Concrete Shear Walls

Introduction

1500 Mission is a project being developed by Related California with guidance from the City of San Francisco and consists of a 39-story residential tower and a 16-story office tower that totals approximately 1.3 million square feet. SOM is the design architect for the entire site and executive architect for the office tower. HKS is the executive architect for the residential tower. DCI Engineers is the structural engineer for the entire project.

The site is located at 1500 Mission Street in San Francisco, California. It is bounded by South Van Ness Avenue to the west, Mission Street to the south, 11th Street to the east, and the One Van Ness building immediately to the north.

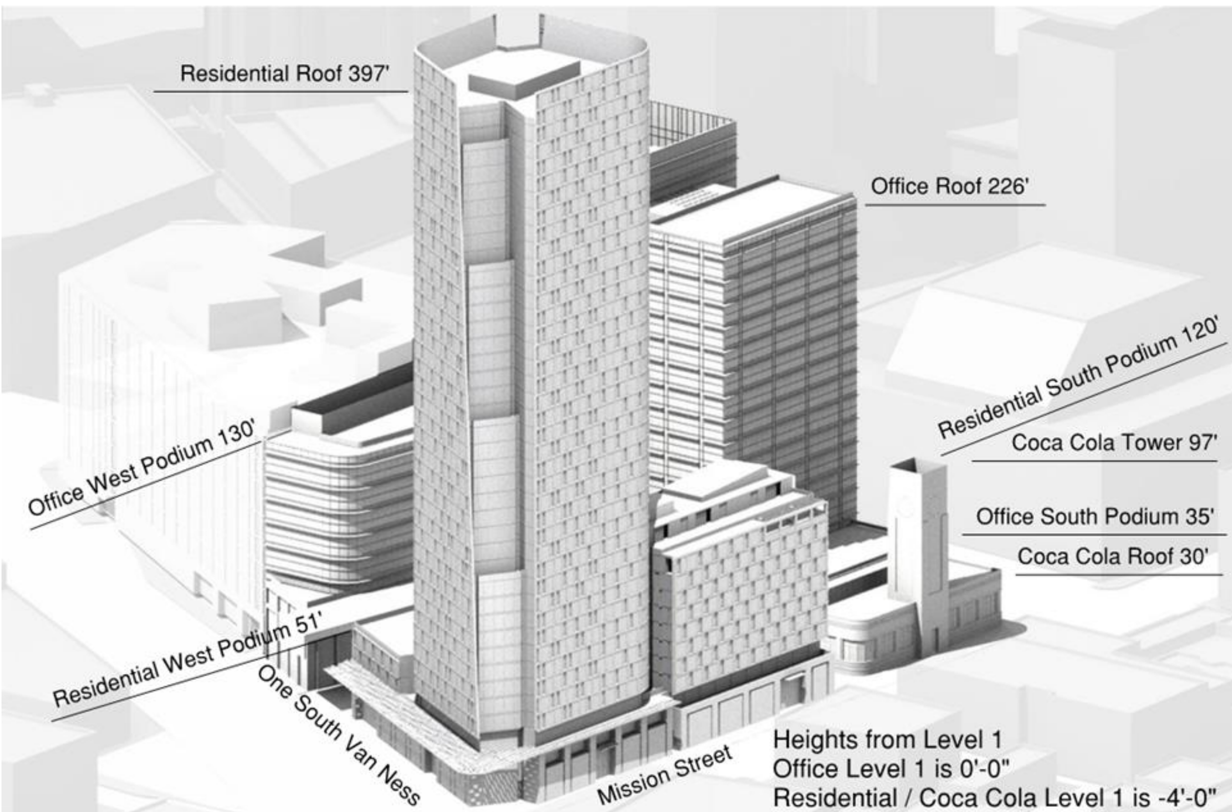
The residential tower is a 39-story, post-tensioned concrete structure with two extended podium wings. The wing bordering South Van Ness is 4-stories tall and the wing along Mission Street is 11-stories in height. A one-story retail space connects the wings and the tower and extends inwards towards the center of the site. Residential amenity space will reside on the roof of that center space. A concrete shear wall core is used to resist lateral forces in the tower along with additional straight concrete shear walls in the podium wings.

The office tower is a 16-story, steel framed structure with concrete on metal deck, also with two podium wings. A nine-story wing borders the adjacent One Van Ness building to the north and a two-story wing is planned to border 11th Street to the east. A concrete shear wall core and c-shaped wall is used to resist lateral forces in the tower along with an additional concrete straight shear wall in the nine-story podium wing.

The two towers are structurally separated above the ground level, but tied together at the ground level and both of the two basement levels.

The existing historic building at the corner of 11th Street and Mission Street (Coca Cola Building) is also being preserved and architecturally integrated into the office tower eastern wing design. The two buildings will remain seismically separated.

An early rendering diagram, curtesy of SOM, showing the relative heights of the project is shown below.



Lateral Systems

For buildings over 240ft in height, a dual system (shear walls and moment frames) are the standard solution unless an alternate approach is used. Alternate systems can be used provided they meet the intent of the code per section 11.1.4 of ASCE 7-10 and are reviewed by a structural design review team. For this project, we chose to use a concrete shear wall only system using a performance based design methodology for both buildings. Dr. Farzad Naeim, Professor John Wallace, and Dr. Shahriar Vahdani are the members of the review team.

Even though the office tower is only 226ft and below the height threshold, the City of San Francisco requested that it be designed with this methodology. Further, the office building is being designed as a Type 3 building at the City's request, which results in an increase in the initial seismic forces that the building is design to by 25% and decreases the allowed drift by 20%, resulting in a more resilient building for more frequently occurring earthquakes.

The 397ft tall residential building, conversely, is being designed as a Type 2 building with no increase in force or decrease in drift.

Both buildings will be designed to not only meet code seismic force requirements (500 year return period), but will also be designed for resultant wind tunnel loading, for a serviceability earthquake event (43 year return period), and a suite of response histories scaled to a Maximum Considered Earthquake spectrum (approximate 2500 year return period). For the non-linear analysis, a conditional mean spectra approach is being used with 11 pairs of response histories scaled to the first mode period and another 11 pairs scaled to the second mode period for each building. Langan Treadwell Rollo is providing the earthquake response histories.

As an interesting aside, the shape and configuration of the residential building were significantly impacted by maintaining the City of San Francisco's wind ordinance, which requires that existing condition total yearly hourly wind speed exceedance over pedestrian comfort levels at the street is not increased with the new development. BMT Fluid Mechanics provided the wind tunnel analysis.

Rendering of the project looking east along Mission Street

Materials

Each building uses a concrete shear wall system to resist lateral forces. Coupling beams within the core walls are composite concrete reinforced with structural steel sections embedded into the shear walls. The latest research by Professor Wallace is incorporated into the analysis for modeling behavior and strength and detailing requirements. Structural steel sections are ASTM A992 Grade 50 with expected strength per AISC of 55 ksi.

The residential building shear walls use 7,000 psi and 10,000 psi nominally specified concrete strength. And the office building uses 8,000 psi nominally specified concrete strength. Expected properties equating to 1.3 times the specified properties were used for all of the analyses. Material data was gathered to justify these values.

Reinforcing within the shear walls is a mixture of ASTM A706 Grade 60 and Grade 80 reinforcing. Confinement ties are 80 ksi while shear reinforcing is 60 ksi. Both Grade 60 and Grade 80 are used for longitudinal reinforcing with the configuration depending upon the most economical layout. Grade 80 was chosen to reduce shear wall congestion, improve sustainability, and create savings for the project. Justification for its use are discussed below.

Nonlinear Modeling

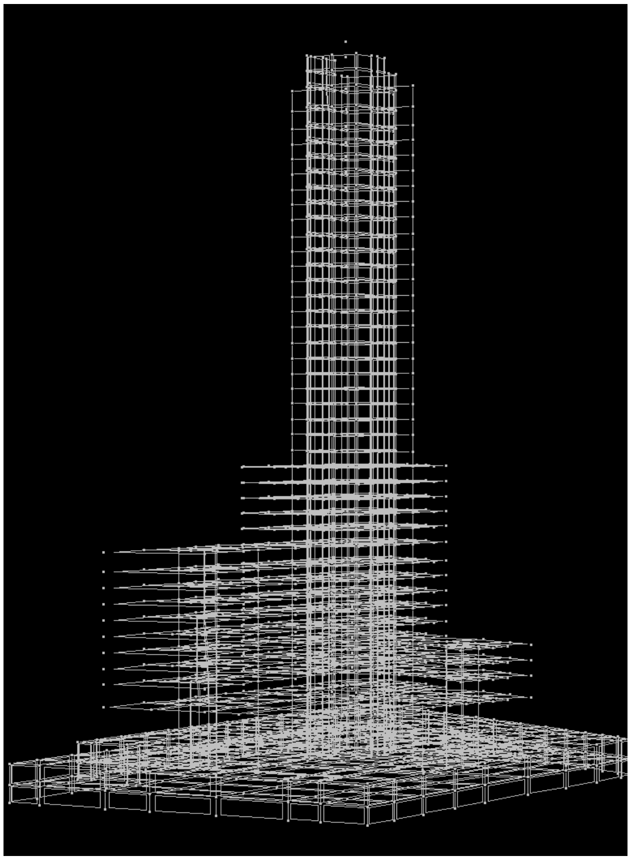
Perform-3D is being used for the nonlinear modeling and response history analysis. Models include the shear wall concrete, reinforcing, lumped gravity columns in proximity to shear walls, and floor framing elements. At rigid diaphragms, single floor framing elements are used on each side of the core. Multiple semi-rigid shell elements, in addition to single floor framing elements, are used at diaphragms near discontinuities or with diaphragms with multiple shear walls. Single elements on each side of the core wall are modeled as elastic elements with lumped plastic hinges at the ends representing the rotational capacity of the joint.

For the upper half of the residential building, significantly above the top of the podium level where the first wing wall is introduced, rigid diaphragms are modeled. Below this, semi-rigid diaphragms are modeled to account for variable diaphragm stiffness.

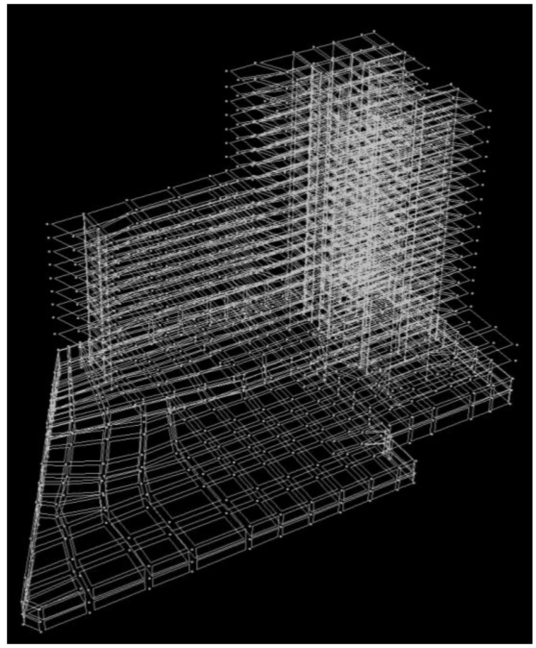
For the office building, the entire building uses semi-rigid diaphragms and shell elements to account for the variable diaphragm stiffness. Composite steel beams are lumped together and modeled as one element on each side of each wall element to account for gravity framing outrigger effects.

High Strength Reinforcing Background

The structural engineering community is always looking for new materials that can improve sustainability as well as provide for more economical buildings. High strength reinforcing is no exception. Reinforcing has evolved from Grade 33 to 40 to 50 to 60 to 75, and now 80 and even 100 and 150 ksi steels. Significant study, testing, and evaluation of ASTM A706 Grade 80 for use in seismic systems is now underway[4]. The NIST report on the "Use of High-Strength Reinforcement in Earthquake-Resistant Concrete Structures"[1] and the material testing report for "Defining Structurally Acceptable Properties of High-Strength Steel Bars through Material and Column Testing"[5] by the Charles Pankow Foundation are two of the references used as the basis for considering ASTM A706 Grade 80 reinforcing in concrete shear walls for this project.



Residential Building Perform-3D Model



Office Building Perform-3D Model

ASTM Grade 80 Longitudinal Reinforcing

ASTM Grade 80 reinforcing used as longitudinal reinforcing in concrete shear walls will reduce material, which will allow for less congestion, which in turn will be easier to construct. Each of these benefits also contribute to a more economical structure and will reduce the overall construction cost.

The decision to use this material for specific projects must be based on availability and testing. Several mills are producing this material as is shown by the gathered test data from these mills in the next section. And shear wall testing at the University of Washington[2] with reinforcing yield strengths varying from 51 to 84 ksi did not show much difference between the materials.

Based on the significant interest in using this material, the availability, and the testing underway, DCI Engineers felt it was the right time to use A706 Grade 80 as longitudinal reinforcing in a real building design. As it is not currently allowed in the building code for this purpose, how and on what basis, became part of the focus of our project basis of design discussion with our structural design review team. With the guidance of Professor John Wallace and Dr. Farzad Naeim, we were able to develop a road map for acceptance of this material for this particular project.

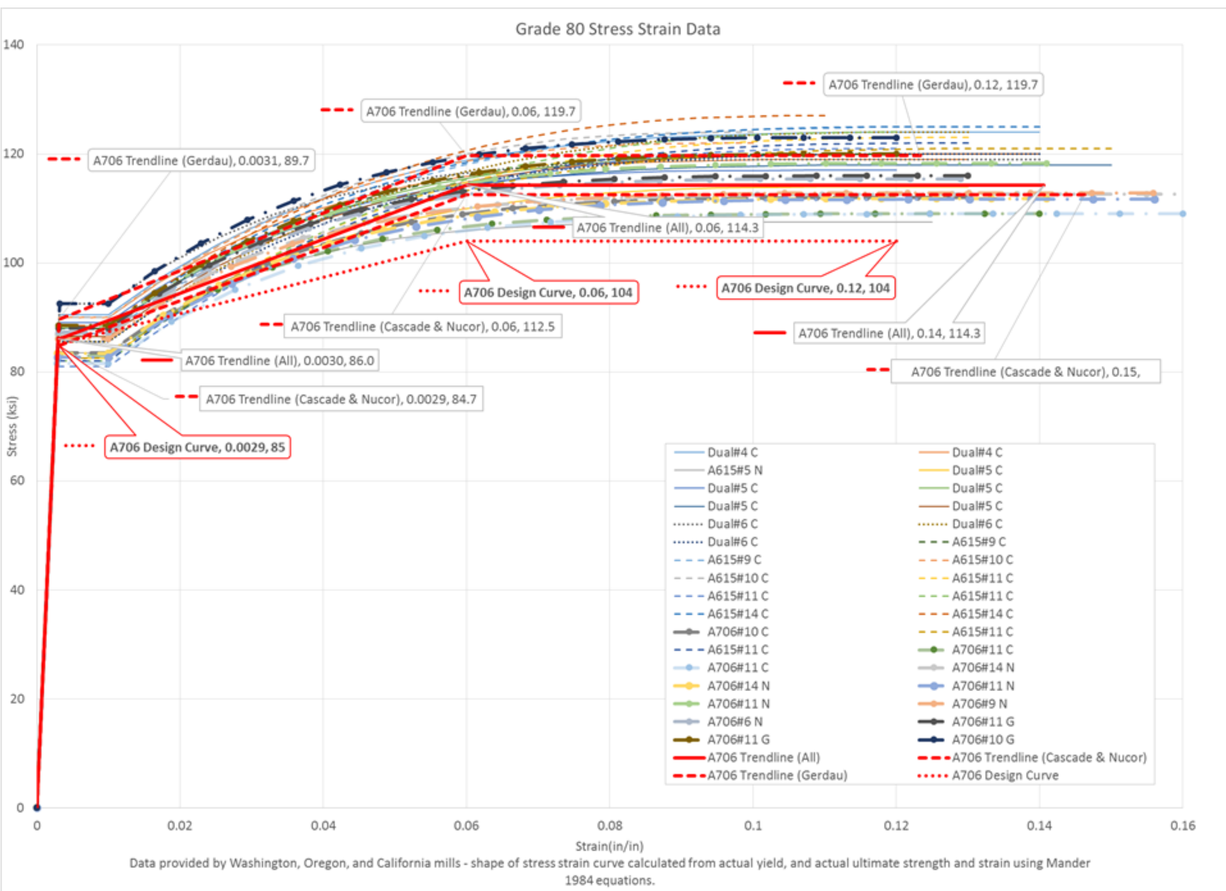
Road Map

The road map for using ASTM A706 Grade 80 for the longitudinal reinforcement in the shear walls includes expected yield and ultimate strengths based on test data, minimum ultimate to yield ratio, strain criteria limits, spacing of confinement reinforcing, ductility requirements for the bars, as well as how the bars are to be spliced.

Material Test Data

Several mill certificates were obtained from Nucor in Seattle, Cascade in McMinnville, Oregon, and Gerdau in Rancho Cucamonga, California. Some of the data is dual certified for A615 and A706 which has fairly wide spread data. The dual graded rebar is for the smaller #4 and #5 bars which will not be used as vertical reinforcing. The dual graded #6 bar trends on the higher ultimate strength side. Some of the data is for A615, which also generally trends up on the higher range. And some is A706. The A706 rebar is from #6, #9, #10, #11, and #14 bar tests.

The overall test data graph is shown below. A trend line developed for all of the A706 rebar, plotted below, closely aligns with the NIST report data from Seattle. Two other trend lines are also plotted. The one for combined Oregon (Cascade) and Washington (Nucor) data closely aligns with the overall data. The trend line for the California (Gerdau) data is a little higher. A design curve that represents the chosen modeling parameters for the Grade 80 reinforcing is also shown.



A706 Test data trend line plotted over all gathered Grade 80 test data

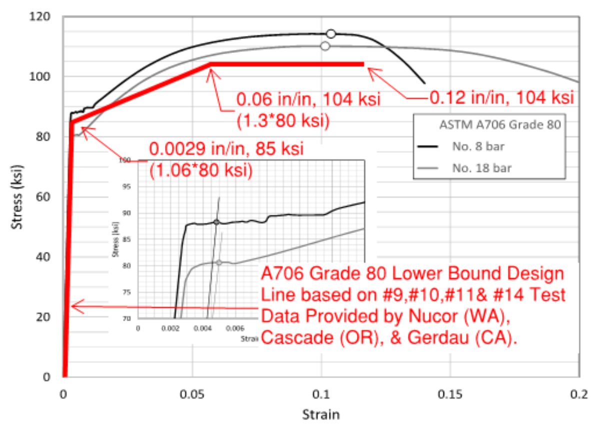


Figure 2-6 Example stress-strain curves for ASTM A706 Grade 80; dots on the curves represent the tensile strength and uniform strain. The inset image shows a larger scale view of where the 2% offset lines cross the stress-strain curves (data courtesy of Nucor Steel Seattle, Inc.).

A706 Grade 80 Design Curve plotted over the NIST report Grade 80 data

Reinforcing is specified such that Fy minimum = 80 ksi and Fu minimum = 1.3(80) = 104 ksi. These are reasonable lower bounds on the data gathered and should be easy to obtain mill certifications for. Fy expected is not specified, but is assumed to be 1.06(80) = 85 ksi based on the average of the test data gathered. We believe this test data, along with data from other reports, supports these conclusions.

Minimum Ultimate Tensile to Yield Ratio

Current code provisions for Grade 60 reinforcing require that the minimum ultimate tensile to yield ratio be at least 1.25. To remain consistent with current practice, the same ratio is being kept for ASTM Grade 80 reinforcing used as longitudinal reinforcing. To ensure that reinforcing being provided correlates well to test data collected, we are also placing a cap on the tensile to yield ratio of 1.5.

Strain Limits

Grade 80 strain at yield is based on Fy/E with E = 29,000 ksi. Strain at the plateau is taken as the same as Grade 60 which is 6%. Data further supports a final fracture elongation of 12%, similar to Grade 60. Based on consensus from the "Tall Building Design Criteria Meeting"[6] in San Francisco dated May 29, 2013, mean reinforcing steel strain limits of 0.005 in compression and 0.01 in tension are being used for acceptance criteria. The limits are the same for Grade 60 and Grade 80.

Confinement Spacing

The Pankow Report[5] details low cycle fatigue tests with various bar diameter spacing for confinement, bar strengths, and load histories, and concluded that bar buckling depends on multiple factors. Bar buckling is not just a function of confinement spacing, but also the load history and metallurgy of the bar. Conservative confinement spacing limits will be applied to Grade 80 for this project with the additional caveat that the material be able to achieve the desired low cycle fatigue requirements of the bar noted in the next section. Based on favorable results from the report, confinement of Grade 80 reinforcing uses the minimum of five times the bar diameter or six inches on center. Whereas Grade 60 reinforcing, per code requirements, is spaced at the minimum of six times the bar diameter or six inches on center.

Ductility Requirements

As noted above, the load history of low cycle fatigue tests for reinforcing (how far the bar is cycled through tension and then compression) affects ductility results. Therefore, Grade 80 reinforcing submitted for use in the project is required to demonstrate that the metallurgy used to produce the reinforcing produces bars with adequate properties that can withstand the number of cycles required for this particular project. Fatigue tests are required to cycle through strains equal to 1.5 times the mean used for design which equate to 0.0075 in compression and 0.015 in tension until failure at a bar grip spacing representing a #5 confinement tie at either five times the bar diameter or six inches on center, whichever is smaller. The number of cycles required for acceptance equates to the maximum number of pertinent cycles in any of the earthquake response records analyzed for the project times 1.5. Low cycle fatigue testing shall comply with ASTM A370, ASTM E8, and ASTM STP 465. At least ten tests per bar size are required and the mean results allowed for use.

Splices

Recent tests at the University of Washington[2] have given more insight into splices in the inelastic region of concrete shear walls. In general, per Lowes et al testing, splices move damage in the inelastic range to where they start and stop because of the increased area of reinforcing over the splice length, regardless of the reinforcing strength. Splices also have the potential to increase the required shear force in the wall to obtain the same amount of drift by shifting the yielding up higher into the wall and thus the shear lever arm. In the elastic range, the splices do not cause any issues, but in the inelastic range, they have a pronounced effect on the location of the damage observed in the wall.

The testing used four specimens; the first two used 84 ksi, the third used 51 ksi, and the last used 67 ksi yield strengths for vertical longitudinal reinforcing. The 67 ksi specimen did not use splices. The 84 ksi tests used boundary elements whereas the 51 ksi test used evenly distributed reinforcing. All of them observed similar amounts of overall damage, but the contributions from different mechanisms varied. The higher strength reinforcing did not appear to change the overall behavior of the wall.

All test specimens had lap lengths designed for Grade 60 reinforcing. Actual rebar yield tested was 84 ksi, so there was more concentration of force transfer over a shorter distance in the splices than the code provisions would intend. Even so, the splices did not fail, but concentrations of strain occurred just outside of the splices in these walls.

In hinge zones of walls, per recommendations of Lowes et al,[2,3] couplers are being used (or no splices at all where practical) to evenly distribute yielding over the reinforcing in the hinge zone and reduce the likelihood of concentrated failures.

Elsewhere, outside of plastic hinging areas where reduced strain demands are anticipated, lap splices detailed for 80 ksi yield strength are used.

The NIST report[1] on high strength reinforcement indicates that for confined splices, a minimum Ktr factor of 1.3 be used for Grade 80 splices when calculated in accordance with ACI 318. Commentary in ACI 318 for development lengths, and therefore splice lengths, indicates that a Ktr factor of less than 2.5 tends to represent splitting failures. A Ktr factor greater than 2.5 is not allowed due to the increased possibility of pull out failures. So, for confined lap splices (outside of the hinge zone), a Ktr factor of 1.3 is maintained when calculating splice lengths. The NIST report graph below shows the normalized referenced test values.

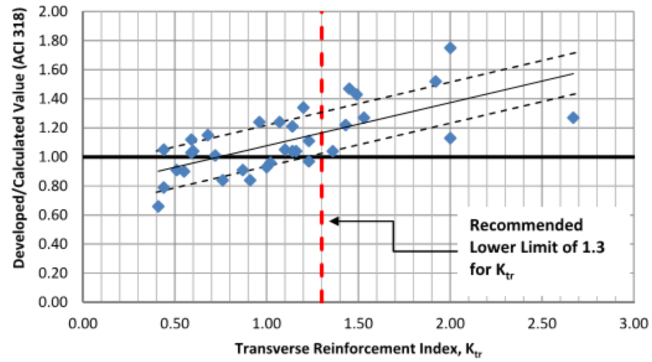


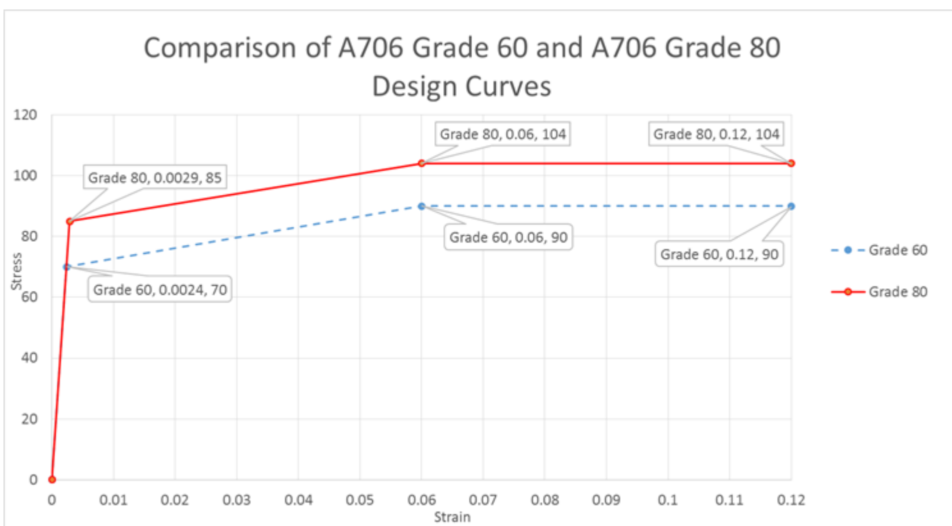
Figure 3-3 Developed-to-calculated stress values versus transverse reinforcement index, K_{tr} (based on data from Seliem et al., 2009).

Taken from NIST report on high strength reinforcement

For unconfined lap splices (again, outside of the hinge zone – typically in the web of the wall), lap splices are designed with a Ktr factor equal to zero (i.e. no confinement) and the term (Cb + Ktr) / db is set to 1.0 for the design of the splice length.

Summary of Recommended Modeling Parameters

- The graph below shows a comparison between A706 Grade 60 and A706 Grade 80 design curves for use in modeling. The mean strain in compression is limited to 0.005 and the mean strain in tension is limited to 0.01



- ASTM Grade 80 Specified Reinforcing: Yield = 80 ksi minimum; Ultimate Tensile = 1.3(80) = 104 ksi minimum; Tensile/Yield ratio = 1.25 minimum and 1.5 maximum; Fracture strain = 12% minimum.

- Grade 80 expected properties assumed but not specified: Yield = 1.06(80) = 85 ksi; Strain at yield = 0.0029 and is based on Fy/E with E = 29,000 ksi; Strain at the plateau is the same as Grade 60 at 6%.

- Grade 80 confinement spacing uses a minimum of five times the bar diameter or six inches on center. Low cycle fatigue tests are required to cycle through strains of 0.0075 in compression and 0.015 in tension until failure at a bar grip spacing representing a #5 confinement tie at either five times the bar diameter or six inches on center, whichever is smaller. The number of cycles required for acceptance equates to the maximum number of pertinent cycles in any of the earthquake response records analyzed for the project times 1.5. At least ten tests per bar size will be required and the mean results allowed for use.

- Splices are avoided or couplers used in the hinge zone. Outside of the hinge zone, lap splices are used. A maximum Ktr factor of 1.3 is used for confined lap splices. A Ktr factor of 1.0 is used for unconfined lap splices.

Future Design

This performance based design project provides a road map for the use of high strength ASTM A706 Grade 80 longitudinal reinforcement in concrete shear walls in seismically active areas. The road map is based upon the use of available research, current test data, sound engineering principles, and material evaluation on a project specific basis. It will serve as part of the foundation for its use and acceptance as longitudinal reinforcement in concrete shear walls as it becomes more commonly used in the future.

References

1) Use of High-Strength Reinforcement in Earthquake-Resistant Concrete Structures (NIST GCR 14-917-30); National Institute of Standards and Technology; March 2014; NEHRP Consultants Joint Venture (Applied Technology Council (ATC) & Consortium of Universities for Research in Earthquake Engineering (CUREE))

2) Earthquake Response of Slender Planar Concrete Walls with Modern Detailing; Laura N. Lowes, Dawn E. Lehman, Anna C. Birely, Daniel A. Kuchma, Kenneth P. Marley, Christopher R. Hart; 2012; Engineering Structures, Vol. 43, pp 31-47

3) Anchorage Detailing Effects on Lateral Deformation Components of R/C Shear Walls; Dissertation; Benton Johnson; 2010

4) Roadmap for the Use of High Strength Reinforcement in Reinforced Concrete Design (ATC-115); Applied Technology Council, 2014 (Final document dated Feb. 27, 2015)

5) Defining Structurally Acceptable Properties of High-Strength Steel Bars through Material and Column Testing, Part 1: Material Testing Report; Charles Pankow Foundation; The University of Texas at Austin, Chase M. Slavin, Wassim M. Ghannoum; August 27, 2015

6) Tall Building Design Criteria Meeting Minutes; Ron O. Hamburger; May 29, 2013