The Impact of High-Strength Reinforcing Steel on Current Design Practice

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INTRODUCTION

The research presented here seeks to assess the potential for high strength reinforcement (HSR) to reduce the volume of reinforcement used in construction of typical concrete buildings and, thereby, reduce construction time, material costs and, ultimately, the total cost of building construction. A series of reinforced concrete building components (beams, slabs, columns, and walls) were designed for typical building geometries and loads, using the current ACI Code (ACI Com. 318 2011), and considering reinforcement design strengths ranging from 60 to 120 ksi. The results of these parallel designs were compared to determine the reduction in reinforcement achieve by employing HSR in place of Grade 60 reinforcement and to establish practical limits on steel strength beyond which steel volume does not diminish with increasing steel strength. Additionally, Code requirements controlling each component design were identified with the objective of determining those that have the greatest impact on design with HSR.

RESEARCH PROCESS

Construction documents for a series of mid- to high-rise reinforced concrete buildings designed for construction on the West Coast in the last 5 years were reviewed to determine the relative volume of reinforcement attributed to beams, slabs, columns, and walls comprising the gravity and lateral load resisting systems. For buildings of this type, it was determined that HSR has the greatest potential for reducing building construction costs if employed in gravity load resisting components (beams, one-way slabs, two-way slabs, and columns) and walls designed to resist lateral loads. This building review also provided typical geometries and loads for use in component designs.

Using the results of the review, designs were developed for one-way slabs, two-way slabs, gravity load resisting beams, gravity columns, and lateral load resisting walls, for reinforcement design strengths ranging from 60 to 120 ksi. Designs utilizing Grade 60 reinforcement were developed to be compliant with the ACI Code. Designs utilizing HSR were developed to be compliant with the ACI Code, with the following three exceptions. First, Code limits on steel yield strength were ignored. Second, beams and slabs containing HSR were not designed to meet ACI Code requirements intended to limit flexural crack widths (Section 10.6). To assess the impact of these requirements, maximum flexural crack widths were computed and compared with Code intended limits and one-way slab designs meeting Code intended limits were developed and compared with those not meeting Code limits. Third, columns utilizing HSR were not designed to meet Code requirements intended to prevent reinforcement yielding due to creep and shrinkage (Section 10.9). For all components considered in the study, reinforcement volumes and the Code requirements controlling the design were compared for designs utilizing different strength reinforcement. Data from these comparisons were used to (1) assess the potential for HSR to reduce steel volume and construction cost and (2) identify Code requirements that warrant further evaluation, given their impact on design with HSR.

BUILDING CONFIGURATIONS AND LOADS

No single reference building was used to support the component design effort. Instead, a range of component geometries and loads typical of mid- to high-rise concrete construction on the West Coast were considered. Beams and slabs were assumed to be part of a continuous system with span lengths between supports ranging from 24 ft. to 30 ft.; a story height of 12 ft. was assumed for column and wall designs. Slab thickness and beam depth are typically determined to meet limits on allowable deflection under service-level loading; here slab thicknesses ranging from 7 in. to 13 in. and beam depths ranging from 22 in. to 30 in. were considered. Columns were assumed to be prismatic with dimensions ranging from 24 in. to 36 in. A rectangular wall 20 ft. long by 2 ft. thick was considered. Gravity loads of 10 psf super imposed dead load and 40 psf live load were used. Reinforced concrete was assumed to have a unit weight of 150 lb/ft^3.

SLABS

One- and two-way slab systems were designed assuming no post-tensioning and considering three- and four-span continuous systems. One-way slabs were designed considering the continuous four-span system shown in Figure 1. The one-way slab section was designed for Sections A-D in Figure 1, which are located at the points of maximum positive and negative flexural demand for each span in half of the symmetric system. For two-way slabs, designs were developed considering a three bay by three bay configuration (Figure 2). Using the design strip approach, column and middle strips were designated and reinforcing steel was distributed appropriately. Table 1 lists span lengths and slab thicknesses for which designs were completed.

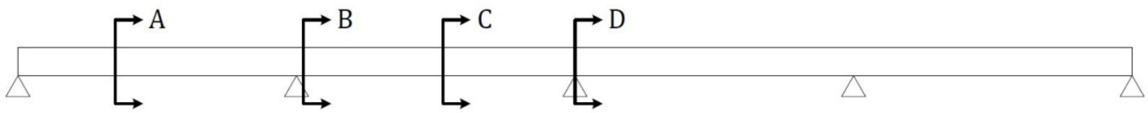


Figure 1 – One-Way System Configuration

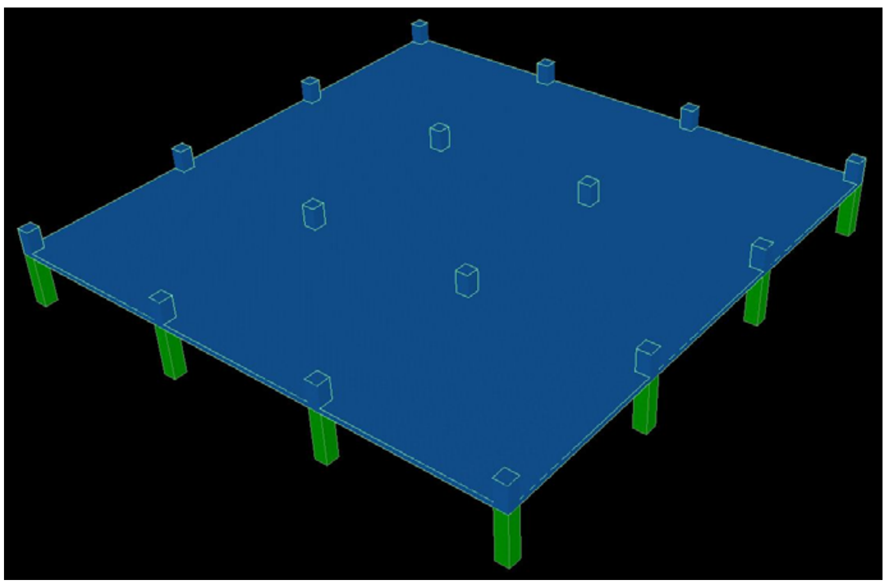


Figure 2 – Two-Way System Configuration

Table 1 – Slab Parameters

Concrete Strength	5,500 psi					
Steel Strength	60, 80, 120 ksi					
Span Length	24 ft.		27 ft.		30 ft.	
Slab Thickness	**One-Way**	**Two-Way**	**One-Way**	**Two-Way**	**One-Way**	**Two-Way**
	7, 8, 9 in.	7, 8, 9 in.	8, 9, 10 in.	9, 10, 11 in.	10, 11, 12 in.	11, 12, 13 in.

BEAMS

Beams were designed considering a continuous three-span system (Figure 3). No post-tensioning was considered. Beams were assumed to be part of a beam-slab system and have a t-shaped cross-section (Figure 4). A beam width of 18 in., slab thickness of 8 in., and slab reinforcement ratio equal to the ACI Code minimum of 0.0018 was assumed for all designs (Section 7.12.2.1). The effective beam flange width was determined based on ACI Code Section 8.12. Beam sections were designed for Sections A-C in Figure 3, which are located at the points of maximum positive and negative flexural demand for each span in half of the symmetric system. Table 2 lists the span lengths and beam depths for which designs were completed.

Table 2 – Beam Parameters

Concrete Strength	5,500 psi		
Steel Strength	60, 80, 120 ksi		
Span Length	27 ft.	30 ft.	30 ft.
Beam Depth	22 in. 24 in. 26 in.	26 in. 28 in. 30 in.	28 in. 30 in. 32 in.

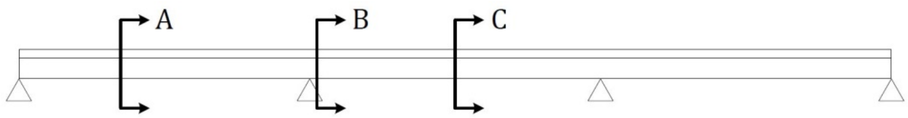


Figure 3 – Beam System Configuration

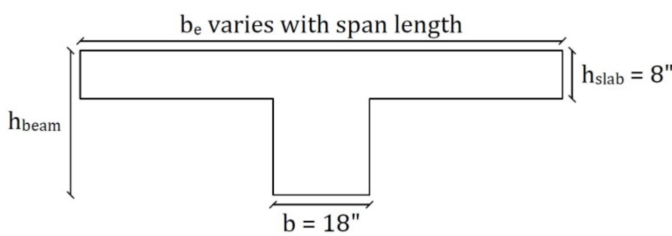
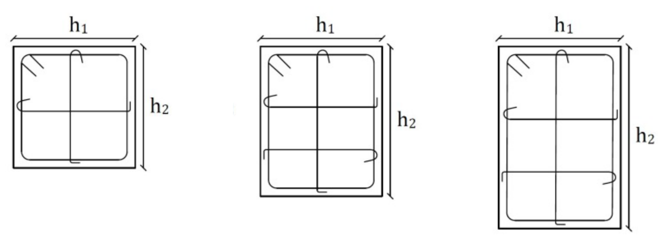


Figure 4 – Beam Cross-Section

Gravity Columns

Columns were assumed to be square or rectangular in shape with a minimum dimension of 24 in. and a story height of 12 ft. Figure 5 shows typical column cross sections, and Table 3 lists dimensions and transverse reinforcement configurations considered in the design process.



Column A Column B Column C

Figure 5 - Column Cross Sections

Table 3 – Column Parameters

Concrete Strength	6,000, 10,000 psi				
Steel Grade	60, 80, 100, 120 ksi (longitudinal) 60, 70, 80, 90, 100, 110, 120 ksi (transverse)				
Geometry	**Column**	h_1	h_2	#ties h_1 dir.	#ties h_1 dir.
	A	24 in.	24 in.	1	1
	B	24 in.	30 in.	1	2
	C	24 in.	36 in.	1	2

STRUCTURAL WALLS

A rectangular wall 20 ft. long and 2 ft. thick with 3 ft. long boundary elements at each end (Table 4 and Figure 6) was considered. A story height of 12 ft. was assumed. Longitudinal reinforcement was concentrated in the boundary elements, and boundary element longitudinal reinforcement ratios were varied to meet strength requirements. The longitudinal reinforcement ratio in the web region was held constant at 0.005 for all designs.

Table 4 - Structural Wall Parameters

Concrete Strength	6,000, 10,000 psi		
Steel Strength	60, 80, 100, 120ksi		
Wall Dimensions	**Length**	**B.E. Length**	**Width**
	20 ft.	3 ft.	2 ft.

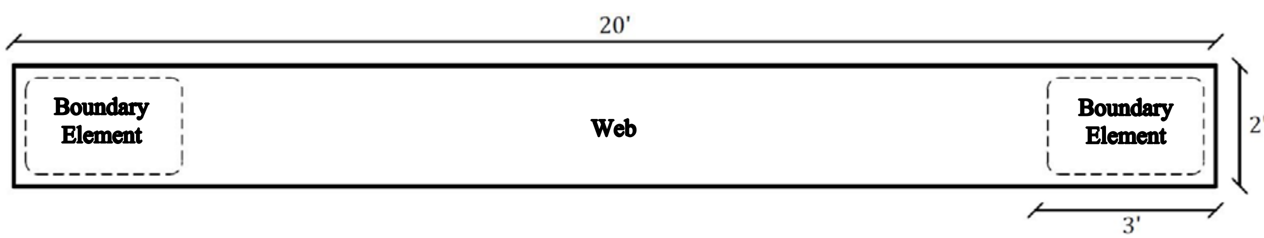


Figure 6 - Wall Cross-Section

LOADING

Slab and beam reinforcement, column transverse reinforcement, and wall horizontal reinforcement was designed to meet strength requirements under design-level loading. Table 5 lists the loads used to determine member demands for design. For design of beams, a tributary width of 24 ft. was used. For design of column and wall longitudinal reinforcement, sections were initially designed using Grade 60 reinforcement and typical longitudinal reinforcement ratios; sections were then re-designed using HSR reinforcement to achieve the same strength as the Grade 60 designs.

Table 5 - System Loading

Component	Super-imposed Dead Load [psf]	Live Load [psf]	Shear Load
Slabs **Beams**	10	40	Consistent with flexural demands
Columns	-	-	Determined by flexural capacity
Structural Walls	-	-	$\{2,4,6,7.5\}A_{cv}\sqrt{f'_c}$ [lb]

DESIGN ASSUMPTIONS AND EVALUATION

Components were designed to meet ACI Code requirements, with three exceptions discussed below. Relevant ACI Code requirements included strength requirements for flexural, shear and axial demands under design-level loading, confinement requirements for columns and walls, and serviceability requirements. Table 6 identifies the ACI Code Sections and other references used in designing and evaluating each component considered in this study.

In three cases, ACI Code requirements were not employed in component design: (1) Code limits on steel yield strength were ignored, (2) beams and slabs utilizing HSR were not designed to meet ACI Code requirements intended to limit flexural crack widths (Section 10.6), and (3) columns utilizing HSR were not designed to meet Code requirements intended to prevent reinforcement yielding due to creep and shrinkage (Section 10.9). For beams and slabs, members were designed to meet ACI Code requirements for strength and for serviceability with respect to cracking associated with shrinkage and temperature loading and local demands; maximum flexural crack widths were assessed and compared with ACI Code intended limits. To assess the impact on steel volume of ACI Code limits on flexural crack widths, one-way slabs were redesigned to meet these limits. For columns and walls, P-M interaction curves, defining the flexural strength of the member cross section for a given axial load, were determined for Grade 60 designs with typical reinforcement ratios; longitudinal reinforcement designs using HSR were then developed to achieve similar P-M interaction curves. The ACI Code minimum reinforcement ratio was chosen as the benchmark reinforcement ratio for columns. This resulted in columns designed using HSR having reinforcement ratios lower than the Code minimum.

Table 6 – Strength, Detailing, and Serviceability Requirements Considered in Design

Component	Flexure	Axial	Shear	Confinement	Deflection	Flexural Crack Width	Shrinkage & Temperature Crack Width	Other
One-way Slabs	10.3		11.2		9.5	10.6 & Frosch	7.12.2	
Two-way Slabs	10.3		11.2		9.5	10.6 & Frosch	7.12.2.1	13.3.2
Beams	10.3		11.2 - 11.4		9.5	10.6 & Frosch		10.5.1
Columns	10.3 & PM	10.3 & PM	11.4 & 21.13	21.6				
Walls	10.3 & PM	10.3 & PM	21.9	21.6				

CONCRETE AND STEEL MATERIAL PROPERTIES USED IN DESIGN

Components were composed of concrete and reinforcing steel. Material properties employed in component designs are listed in Tables 1-4. A concrete compressive strength of 5,500 psi was assumed for slabs and beams; compressive strengths of 6,000 psi and 10,000 psi were assumed for design of walls and columns. Concrete material response was characterized per Section 10.2 of the ACI Code. This included a usable compressive strain limit of 0.003 and use of the Whitney stress block and the assumption of zero tensile strength for calculation of flexural strength. For each component, designs were completed for Grade 60 reinforcement, which was assumed to conform to either ASTM A615 or ASTM A706. Additional designs were completed assuming reinforcement yield strengths ranging from 70 ksi to 120 ksi. For design, all reinforcing steel was assumed to exhibit elastic-perfectly-plastic stress-strain response and maintain the specified yield strength to the required strain demands. With the exception of column shear design, which is discussed in detail below, components were not capacity designed, thus the potential for reinforcing steel to develop strength in excess of the yield strength was not considered.

DESIGN FOR FLEXURAL DEMANDS

For beams and slabs, longitudinal tension reinforcement was designed to meet flexural strength requirements for design-level loading using the iterative process outlined in Figure 7. In Figure 7, M_u is the ultimate (factored) flexural demand on the section (ACI Code Section 9.2) determined by design-level loading (Table 5), f_y is nominal yield strength of the reinforcement, ϕ = 0.90 for flexural design of tension-controlled sections, d is the depth to the centroid of tension reinforcement, a is the depth of the equivalent rectangular stress block (ACI Code Section 10.2.7.1), A_s is the area of tension reinforcement, b is the width of the compression section, and M_n is the flexural capacity of the section (ACI Code Sections 10.2 and 10.3). In evaluating flexural strength, compression reinforcement was ignored, resulting in a slightly conservative estimate of flexural strength. Once longitudinal reinforcement was finalized, the cracked moment of inertia was computed for use in deflection calculations.

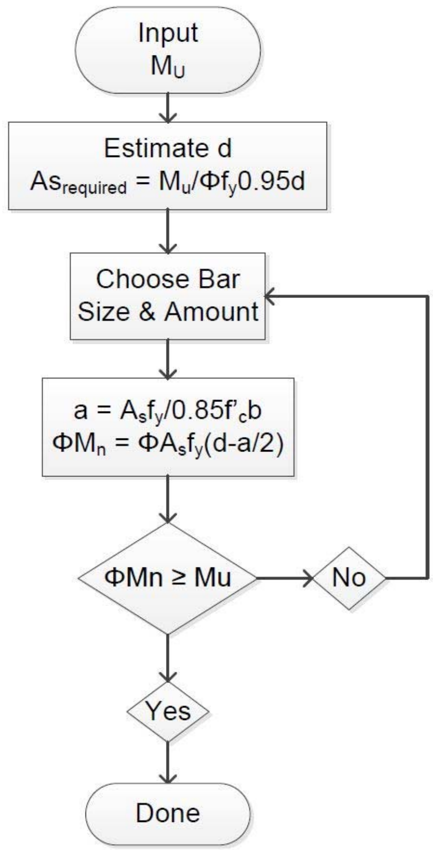


Figure 7 - Flexural Design Flow

EVALUATION OF COLUMN STRENGTH FOR FLEXURAL AND AXIAL LOADING

To compare column designs for Grade 60 and HSR, for each column geometry (Figure 5) a reference column section was designed employing Grade 60 reinforcement and a longitudinal reinforcement ratio of approximately 1.0%. The software spColumn (http://www.structurepoint.org) was used to determine a P-M interaction curve defining flexural strength as a function of axial load. The spColumn software was developed by the Engineering Software Group of the Portland Cement Association (PCA) and defines flexural and axial strength per the ACI Code. The reference column was then redesigned using HSR with different yield strengths to achieve a factored flexural strength approximately equal to that of the Grade 60 design for axial load ratios ranging from 10% to 30% of the factored axial strength. Thus, HSR column sections had longitudinal reinforcement ratios less than 1.0%. P-M interaction curves were created for the column designs utilizing HSR. A similar process was used for walls.

DESIGN OF TRANSVERSE REINFORCEMENT

Transverse reinforcement was designed to meet shear strength and confinement requirements, as appropriate. For slabs and beams, transverse reinforcement was designed to meet shear strength requirements (ACI Code Sections 11.1-11.4). For two-way slabs, design for punching shear at column supports (Section 11.11) was not considered. For beams, shear capacity requirements were met through use of stirrups; in beam regions where shear demand was minimal, minimum shear reinforcement was provided. For slabs, shear demands were such that stirrups were not required. For columns, transverse reinforcement was provided to meet confinement and shear requirements. Design for confinement was controlled by Sections 7.10.5 and 21.9.6. Design for shear employed shear demands associated with maximum probable flexural strength, M_{pr}, per Section 21.5.4 and capacity defined by Section 11.4. If shear demand was not found to govern design, transverse reinforcement was provided per the confinement requirements of 7.10.5 and 21.9.6. For structural walls, boundary element transverse reinforcement was designed to meet confinement requirements (Section 21.9.6), and web horizontal reinforcement was designed to meet shear strength requirements (Sections 11.4 and 21.9.4).

DEFLECTION EVALUATION

Slab and beam designs were analyzed to determine maximum deflections under sustained service-level loads. Deflections were calculated using elastic analysis with reduced section stiffnesses per Figure 8. In Figure 8, I_G is the gross section moment of inertia based on the untransformed concrete section, y_{bar} is the neutral axis depth corresponding to I_G, M_{cr} is the moment found at cracking of the member, y_{cr} is the neutral axis depth after cracking, I_{cr} is the moment of inertia after cracking based on the transformed section, $M_{service}$ refers to either M_D (demand under service dead conditions) or M_{D+L} (demand under service dead and live conditions), Δ_D and Δ_{D+L} are the deflections found under service-level dead and service-level dead plus live load, respectively, and Δ_{TOT} is the total deflection under sustained service-level loads. The analysis process outlined in Figure 8 is described below for the continuous beam system:

i. For each beam section in the continuous system, the moment of inertia of the transformed cracked section was computed (I_{cr}). As with the section designs, it was assumed that compression reinforcement had negligible impact on the stiffness.

ii. For each section of the continuous system ACI Code Eq. 9-8 (repeated as Eq. 1 below) was used to compute the effective moment of inertia, I_e, for the dead load moment demand and the dead plus live load moment demand:

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

where M_a is the moment demand and all equations variables are as previously defined.

iii. For each span of the continuous beam, an average moment of inertia was computed for use in deflection calculations. This average moment of inertia was defined equal to the weighted average of the effective moments of inertia (Eq. 1) for each section design composing the span. The contribution of each section to the weighted average was determined by the moment distribution and the portion of the span over which each section design could be expected to extend. The ratio of the average moment of inertia to the gross moment of inertia was used with the analysis software to determine the final deflection under service-level loads.

iv. To determine deflections under sustained service-level loading ACI Eq. 9-11 (repeated as Eq. 2 below) was used:

$$\lambda_\Delta = \frac{\xi}{1+50\rho'} \qquad \text{Eq. 2}$$

where ξ is the time-dependent factor defined in ACI Code Section 9.5.2.5 and ρ' is the compression reinforcement ratio, taken as zero in this study. This λ_Δ is then multiplied by Δ_D and added to Δ_{D+L} to define Δ_{TOT}.

To determine the acceptability of designs, maximum deflections computed using the above process were compared with ACI Code limits (ACI Code Table 9.5) and a limit of 0.75 in., commonly employed in practice.

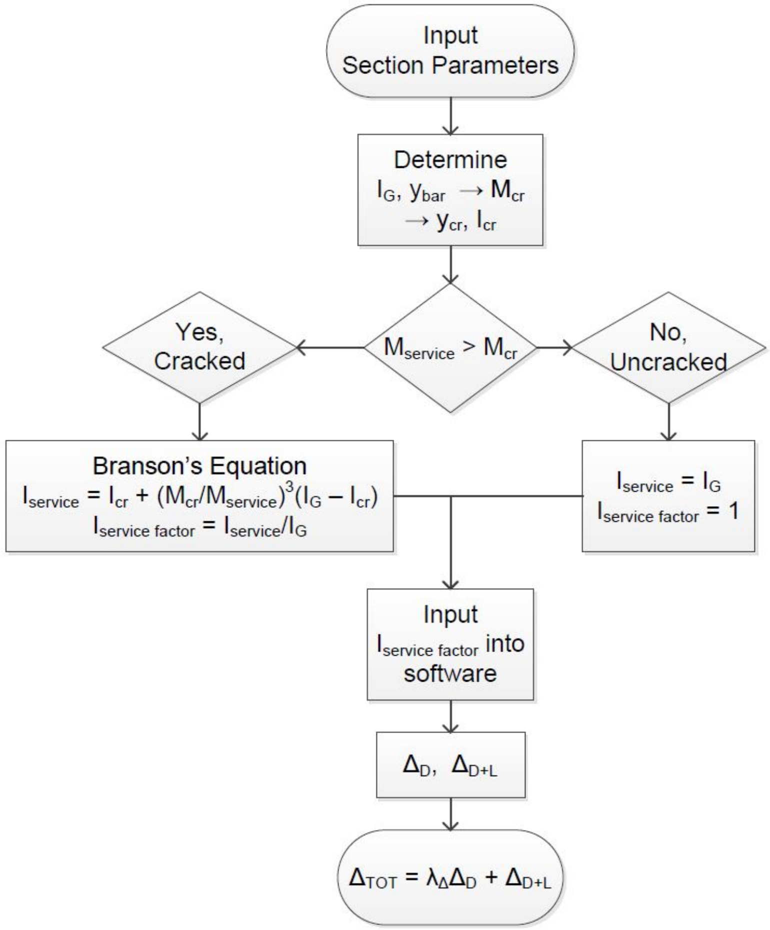


Figure 8 - Deflection Evaluation Flow

CRACK WIDTH EVALUATION

To determine flexural crack widths, the approach outlined by Frosch (1999) was used. After the reinforcement within bending sections was designed, the following equations were used to determine the crack width that would form under service conditions.

$$w_c = 2\frac{f_s}{E_s}\beta d^* \qquad \text{Eq. 3a}$$

where

$$\beta = 1.0 + 0.08d_c \qquad \text{Eq. 3b}$$

$$d^* = minimum\left\{\sqrt{d_c^2 + d_s^2},\ \sqrt{d_c^2 + \left(\frac{s}{2}\right)^2}\right\} \qquad \text{Eq. 3c}$$

and f_s is the stress under service-level loads, d_c is the vertical clear cover, d_s is the horizontal clear cover, and s is the bar spacing for longitudinal reinforcement.

The acceptance criterion for crack widths was taken to be 0.018 in. per the commentary of ACI Section 10.6.4 and Harries et al. (2012).

ONE-WAY SLABS

A series of one-way slabs were designed to meet ACI Code requirements for gravity loading. A continuous four-span system was assumed (Figure 1), and span lengths ranging from 24 ft. to 30 ft. were considered (Table 1). Designs were completed for Grade 60 and f_y = 80 ksi and 120 ksi mild reinforcement (Table 1). For each span length, section designs were completed to meet flexural strength requirements for multiple slab thicknesses. Deflections under service-level loading were computed using elastic analysis with effective stiffnesses defined per Eq. 1. Long-term deflections were determined using Eq. 2. For each section design, flexural crack widths were computed using Eq. 3. Slab section designs meeting ACI Code and commonly employed deflection limits were identified. For designs meeting deflection requirements but not ACI Code limits for flexural crack widths, slab reinforcement was redesigned to meet crack width requirements. In all cases, shear demand was such that the concrete shear strength was adequate and no vertical reinforcement was required.

Table 7 provides details of the one-way slab designs. These include, for each slab section designed, the tension reinforcement configuration and ratio (ρ), the ratio of flexural capacity to demand ($\phi M_n/M_u$) and the crack section effective stiffness (I_{cr}/I_G) as well as, for exterior and interior spans, maximum long-term deflection (Δ_{TOT}), the effective stiffnesses used for deflection calculations (I_D/I_G and I_{D+L}/I_G) and maximum flexural crack width (w_c). In Table 7, designs meeting ACI Code intended flexural crack width or standard practice deflection limits are identified, and data for designs meeting both limits are presented in bold font. Figure 9 provides a graphical comparison of designs utilizing different strength steels for the case of 27 ft. spans and a 10 in. slab thickness. Data in Figure 9 include, for each steel strength, (1) the total weight of reinforcement required for slab design normalized by the weight required for the Grade 60 design, (2) the maximum slab deflection normalized by the commonly employed deflection limit of 0.75 in., and (3) the maximum flexural crack width normalized by the Code intended limit of 0.018 in.

FLEXURAL DESIGN FOR DESIGN-LEVEL DEMANDS

Flexural design was accomplished considering a one foot wide strip of the continuous one-way slab system. Elastic analysis and the RISA-3D structural analysis software (http://www.risatech.com/) were used to determine moment and shear demands for design. Previously described gravity loads were used. Given the symmetry of the four-span system, four different slab sections were designed (Figure 1): Sections A and B located, respectively, at the points of maximum positive and negative moment in the exterior spans and Sections C and D located, respectively, at the points of maximum positive and negative moment in the interior spans. For 27 ft. spans, a 10 in. slab thickness, and Grade 60 reinforcement, the longitudinal reinforcement ratio along the length of the beam ranged from 0.18% (No. 4 at 11 in.) to 0.40% (No. 4 at 5 in.).

Table 7 provides details of the one-way slab designs. These data show that for many slab sections and configurations, strength requirements control the design and the use of HSR results in a significant reduction in steel area. For sections where design is controlled by strength requirements, the ratio of the areas of HSR to Grade 60 reinforcement required to meet flexural demands is approximately equal to the inverse of the ratio of the yield strengths of the steel. The data in Table 7 show also that Code requirements for minimum reinforcement, intended to limit cracking due to temperature and shrinkage (ACI Code Section 7.12.2.1c), control design at slab sections with low flexural demands (slab Sections C and D), for thicker slabs, and for higher strength steel. For these slab sections and configurations, the use of HSR results in no or a limited reduction in steel area. Note that sections for which minimum reinforcement requirements controlled design are identified in Table 7. Data in Figure 9 show that when the total weight of steel required for slab design is considered, the use of HSR results in a significant reduction in steel volume (~21% for f_y = 80 ksi and ~34% for f_y = 120 ksi); this reduction is not as large as would be expected if design was controlled entirely by strength requirements. Data in Figure 9 represent a particular slab geometry; similar results could be expected for other slab geometries.

Table 7 - One-Way Slab Results

24' Span		Section A				Section B				Section C				Section D				Exterior Span				Interior Span			
		Rebar	ρ	$\varphi M_n/M_u$	I_{cr}/I_G	Rebar	ρ	$\varphi M_n/M_u$	I_{cr}/I_G	Rebar	ρ	$\varphi M_n/M_u$	I_{cr}/I_G	Rebar	ρ	$\varphi M_n/M_u$	I_{cr}/I_G	I_D/I_G	I_{D+L}/I_G	Δ_{TOT} [in]	w_c [in]	I_D/I_G	I_{D+L}/I_G	Δ_{TOT} [in]	w_c [in]
7"	60	#4 @ 7”	0.41%	1.05	0.134	#4 @ 5”	0.57%	1.06	0.177	#4 @ 15”	0.19%	1.21	0.069	#4 @ 8”	0.36%	1.19	0.119	0.91	0.52	-1.01	0.0130[1]	0.89	0.80	-0.15[1]	0.0101[1]
	80	#4 @ 9”	0.32%	1.08	0.108	#4 @ 7”	0.41%	1.01	0.134	#4 @ 16” [2]	0.18%	1.50	0.065	#4 @ 12”	0.24%	1.06	0.084	0.91	0.50	-1.03	0.0204	0.89	0.79	-0.15[1]	0.0178[1]
	120	#4 @ 14”	0.20%	1.05	0.074	#4 @ 10”	0.29%	1.06	0.098	#4 @ 16” [2]	0.18%	2.23	0.065	#4 @ 18” [2]	0.18%	1.19	0.065	0.91	0.48	-1.06	0.0464	0.88	0.78	-0.15[1]	0.0337
8"	**60**	**#4 @ 8”**	**0.31%**	**1.01**	**0.116**	**#4 @ 5”**	**0.50%**	**1.17**	**0.173**	**#4 @ 13”**	**0.19%**	**1.53**	**0.076**	**#4 @ 10”**	**0.25%**	**1.06**	**0.096**	**0.98**	**0.79**	**-0.61[1]**	**0.0150[1]**	**0.97**	**0.84**	**-0.11[1]**	**0.0092[1]**
	80	**#4 @ 10”**	**0.25%**	**1.08**	**0.096**	**#4 @ 7”**	**0.36%**	**1.12**	**0.130**	**#4 @ 14” [2]**	**0.18%**	**1.89**	**0.071**	**#4 @ 14”**	**0.18%**	**1.01**	**0.071**	**0.97**	**0.78**	**-0.61[1]**	**0.0225**	**0.97**	**0.83**	**-0.11[1]**	**0.0161[1]**
	120	#4 @ 14” [2]	0.18%	1.16	0.071	#4 @ 11”	0.23%	1.07	0.088	#4 @ 14” [2]	0.18%	2.80	0.071	#4 @ 14” [2]	0.18%	1.50	0.071	0.97	0.77	-0.62[1]	0.0422	0.97	0.82	-0.11[1]	0.0365
9"	**60**	**#4 @ 8”**	**0.28%**	**1.09**	**0.112**	**#4 @ 6”**	**0.37%**	**1.06**	**0.144**	**#4 @ 12” [2]**	**0.19%**	**1.78**	**0.079**	**#4 @ 11”**	**0.20%**	**1.04**	**0.085**	**1.00**	**0.91**	**-0.44[1]**	**0.0007[1]**	**1.00**	**0.88**	**-0.08[1]**	**0.0115[1]**
	80	#4 @ 11”	0.20%	1.06	0.085	#4 @ 8”	0.28%	1.06	0.112	#4 @ 12” [2]	0.19%	2.37	0.079	#4 @ 12” [2]	0.19%	1.27	0.079	1.00	0.90	-0.44[1]	0.0009[1]	1.00	0.88	-0.08[1]	0.0190
	120	#4 @ 12” [2]	0.19%	1.45	0.079	#4 @ 12”	0.19%	1.06	0.079	#4 @ 12” [2]	0.19%	3.51	0.079	#4 @ 12” [2]	0.19%	1.88	0.079	1.00	0.90	-0.44[1]	0.0010[1]	1.00	0.88	-0.08[1]	0.0400

27' Span		Section A				Section B				Section C				Section D				Exterior Span				Interior Span			
		Rebar	ρ	$\varphi M_n/M_u$	I_{cr}/I_G	Rebar	ρ	$\varphi M_n/M_u$	I_{cr}/I_G	Rebar	ρ	$\varphi M_n/M_u$	I_{cr}/I_G	Rebar	ρ	$\varphi M_n/M_u$	I_{cr}/I_G	I_D/I_G	I_{D+L}/I_G	Δ_{TOT} [in]	w_c [in]	I_D/I_G	I_{D+L}/I_G	Δ_{TOT} [in]	w_c [in]
8"	60	#4 @ 6”	0.42%	1.05	0.149	#4 @ 4”	0.63%	1.13	0.208	#4 @ 13”	0.19%	1.18	0.076	#4 @ 8”	0.31%	1.01	0.116	0.89	0.47	-1.31	0.0116[1]	0.86	0.78	-0.20[1]	0.0083[1]
	80	#4 @ 8”	0.31%	1.05	0.116	#4 @ 6”	0.42%	1.01	0.149	#4 @ 14”[2]	0.18%	1.46	0.071	#4 @ 13”	0.19%	1.07	0.096	0.88	0.44	-1.35	0.0191	0.85	0.76	-0.20[1]	0.0158[1]
	120	#4 @ 12” [2]	0.21%	1.05	0.082	#4 @ 9”	0.28%	1.01	0.105	#4 @ 14”[2]	0.18%	2.16	0.071	#4 @ 14” [2]	0.18%	1.15	0.071	0.88	0.42	-1.40	0.0403	0.85	0.75	-0.20[1]	0.0323
9"	60	#4 @ 6”	0.37%	1.13	0.144	#4 @ 4”	0.56%	1.23	0.202	#4 @ 12” [2]	0.19%	1.38	0.079	#4 @ 8”	0.28%	1.09	0.112	0.93	0.64	-0.86	0.0108[1]	0.91	0.82	-0.15[1]	0.0077[1]
	80	#4 @ 9”	0.25%	1.01	0.101	#4 @ 6”	0.37%	1.10	0.144	#4 @ 12” [2]	0.19%	1.83	0.079	#4 @ 11”	0.20%	1.06	0.085	0.92	0.62	-0.88	0.0220	0.90	0.81	-0.15[1]	0.0147[1]
	120	#4 @ 12” [2]	0.19%	1.133	0.079	#4 @ 9”	0.25%	1.10	0.101	#4 @ 12” [2]	0.19%	2.71	0.079	#4 @ 12” [2]	0.19%	1.44	0.079	0.92	0.60	-0.89	0.0375	0.90	0.80	-0.15[1]	0.0300
10"	**60**	**#4 @ 7”**	**0.29%**	**1.04**	**0.121**	**#4 @ 5”**	**0.40%**	**1.06**	**0.161**	**#4 @ 11” [2]**	**0.18%**	**1.60**	**0.081**	**#4 @ 9”**	**0.22%**	**1.04**	**0.097**	**0.97**	**0.88**	**-0.58[1]**	**0.0008[1]**	**0.97**	**0.85**	**-0.12[1]**	**0.0102[1]**
	80	**#4 @ 9”**	**0.22%**	**1.08**	**0.097**	**#4 @ 7”**	**0.29%**	**1.01**	**0.121**	**#4 @ 11” [2]**	**0.18%**	**2.13**	**0.081**	**#4 @ 11”[2]**	**0.18%**	**1.13**	**0.081**	**0.97**	**0.87**	**-0.59[1]**	**0.0009[1]**	**0.97**	**0.84**	**-0.12[1]**	**0.0179[1]**
	120	#4 @ 11” [2]	0.18%	1.32	0.081	#4 @ 10”	0.20%	1.06	0.088	#4 @ 11” [2]	0.18%	3.15	0.081	#4 @ 11” [2]	0.18%	1.68	0.081	0.97	0.87	-0.59[1]	0.0011[1]	0.96	0.84	-0.12[1]	0.0342

30' Span		Section A				Section B				Section C				Section D				Exterior Span				Interior Span			
		Rebar	ρ	$\varphi M_n/M_u$	I_{cr}/I_G	Rebar	ρ	$\varphi M_n/M_u$	I_{cr}/I_G	Rebar	ρ	$\varphi M_n/M_u$	I_{cr}/I_G	Rebar	ρ	$\varphi M_n/M_u$	I_{cr}/I_G	I_D/I_G	I_{D+L}/I_G	Δ_{TOT} [in]	w_c [in]	I_D/I_G	I_{D+L}/I_G	Δ_{TOT} [in]	w_c [in]
10”	60	#4 @ 5”	0.40%	1.16	0.161	#4 @ 4”	0.50%	1.06	0.195	#4 @ 11” [2]	0.18%	1.27	0.081	#4 @ 7”	0.29%	1.05	0.121	0.89	0.54	-1.15	0.0093[1]	0.87	0.81	-0.20[1]	0.0090[1]
	80	#4 @ 7”	0.29%	1.11	0.121	#4 @ 5”	0.40%	1.12	0.161	#4 @ 11” [2]	0.18%	1.69	0.081	#4 @ 9”	0.22%	1.09	0.097	0.89	0.52	-1.17	0.0164[1]	0.86	0.80	-0.20[1]	0.0127[1]
	120	#4 @ 11” [2]	0.18%	1.06	0.081	#4 @ 8”	0.25%	1.06	0.108	#4 @ 11” [2]	0.18%	2.51	0.081	#4 @ 11” [2]	0.18%	1.32	0.081	0.88	0.50	-1.20	0.0372	0.85	0.79	-0.20[1]	0.0283
11”	60	#4 @ 6”	0.30%	1.03	0.133	#4 @ 4”	0.45%	1.11	0.187	#4 @ 10” [2]	0.18%	1.47	0.085	#4 @ 7”	0.26%	1.10	0.116	0.92	0.70	-0.83	0.0119[1]	0.90	0.83	-0.16[1]	0.0085[1]
	80	#4 @ 8”	0.23%	1.02	0.103	#4 @ 5”	0.36%	1.18	0.155	#4 @ 10” [2]	0.18%	1.95	0.085	#4 @ 10” [2]	0.18%	1.03	0.085	0.92	0.69	-0.83	0.0198	0.90	0.82	-0.16[1]	0.0121[1]
	120	#4 @ 10” [2]	0.18%	1.23	0.085	#4 @ 8”	0.23%	1.11	0.103	#4 @ 10” [2]	0.18%	2.90	0.085	#4 @ 10” [2]	0.18%	1.53	0.085	0.92	0.67	-0.84	0.0297	0.89	0.81	-0.16[1]	0.0270
12”	**60**	**#4 @ 6”**	**0.28%**	**1.07**	**0.127**	**#4 @ 4”**	**0.42%**	**1.17**	**0.180**	**#4 @ 9” [2]**	**0.19%**	**1.71**	**0.089**	**#4 @ 8”**	**0.21%**	**1.01**	**0.099**	**0.96**	**0.89**	**-0.61[1]**	**0.0007[1]**	**0.95**	**0.86**	**-0.13[1]**	**0.0081[1]**
	80	**#4 @ 8”**	**0.21%**	**1.07**	**0.099**	**#4 @ 6”**	**0.28%**	**1.04**	**0.127**	**#4 @ 9” [2]**	**0.19%**	**2.27**	**0.089**	**#4 @ 9” [2]**	**0.19%**	**1.20**	**0.089**	**0.96**	**0.88**	**-0.61[1]**	**0.0009[1]**	**0.95**	**0.85**	**-0.13[1]**	**0.0156[1]**
	120	#4 @ 9” [2]	0.19%	1.42	0.089	#4 @ 9”	0.19%	1.04	0.089	#4 @ 9” [2]	0.19%	3.37	0.089	#4 @ 9” [2]	0.19%	1.78	0.089	0.96	0.87	-0.62[1]	0.0010[1]	0.95	0.84	-0.13[1]	0.0320

Notes: Bold font indicates a design that meets ACI Code and standard practice deflection and crack width limits. 1. Meets ACI Code and standard practice deflection or crack width limits. 2. Design is controlled by ACI Code Section 7.12.2.1c, which specifies a minimum longitudinal reinforcement ratio to control cracking due to temperature and shrinkage.

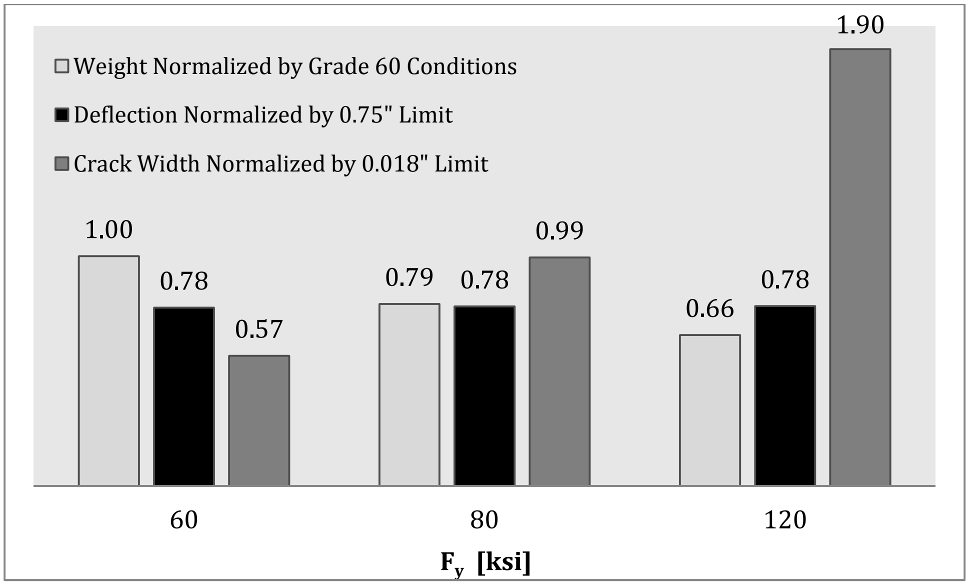


Figure 9 – One-Way Slab Example – 27 ft. Span Length, 10 in. Slab Thickness

EVALUATION OF PERFORMANCE UNDER SERVICE-LEVEL LOADING

The performance of the strength-based designs under service-level loading was assessed on the basis of the i) maximum deflection of the slab and ii) width of flexural cracks orthogonal to flexural reinforcement. Table 7 lists the computed deflections in each span and the stiffness factors used in the calculations. The maximum deflection occurs in the exterior span. Table 7 lists also maximum flexural crack widths; these also occur in the exterior span. Data for slab designs meeting the Code intended flexural crack width limit (0.018 in.) and the deflection limit used commonly in practice (0.75 in.) are present using bold font.

The data in Table 7 show that the deflections are essentially insensitive to steel strength and corresponding reinforcement ratios, with an increase in steel yield strength and reduction in steel area resulting in at most a 7% increase in deflection. This is somewhat unexpected since the reduced area of reinforcing steel results in a lower cracked moment of inertia. From the exterior and interior stiffness factors shown in the table, it can be seen these factors are also relatively insensitive to the steel strength and area. This is due to two factors: i) the slab is either uncracked or loaded only slightly beyond the cracking moment under service conditions, with the result that the deflections are essentially determined by the gross-section moment of inertia, which is not a function of steel area, and ii) the cracked moments of inertia are much smaller than the gross section moments of inertia, with the result that in using (Eq 1) to compute I_{eff}, even in regions where M_a/M_{cr} is large, variation in I_{cr} is mitigated by the significant contribution of I_G and the relatively insignificant contribution of I_{cr} to I_{eff}.

For the scenarios considered, Table 7 presents also the maximum width of flexural cracks forming orthogonal to the longitudinal reinforcement, w_c. The maximum flexural crack width occurs in the exterior span, at either the point of maximum positive (Section A in Figure 1) or negative (Section B in Figure 1) moment, depending on the spacing of longitudinal reinforcement designed for these sections. For HSR, crack widths increase due to i) the higher steel stresses and strains that develop when steel area is decreased and ii) the wider bar spacing associated with reduced steel area. The general trend seen is that crack widths increase as the steel strength increases, for the higher steel strengths this increase causes the widths to exceed the Code intended maximum of 0.018 in. Use of f_y = 80ksi resulted in the crack width limit being exceeded on average by 7% while use of f_y = 120ksi resulted in the crack width limit being exceeded by over 50% on average.

To assess the impact of the crack width limit on designs using HSR, for each span length, the minimum slab thickness meeting the deflection requirement was redesigned to meet the crack width limit as well. Table 8 provides data for the original and redesigned Sections A and B; data are provided only for these sections were crack widths were maximum. Redesign of these sections did not affect deflection calculations. It was found that flexural crack width limits could be met without increasing the volume of HSR. However, this required reducing bar size and bar spacing while increasing the number of bars, which could be expected to increase construction cost. Specifically, the data in Table 8 show that if bar sizes are reduced to No. 3, it is possible to meet flexural crack width requirements without significantly increasing the area of longitudinal reinforcement.

CONCLUSIONS REGARDING DESIGN OF ONE-WAY SLABS USING HSR

The use of HSR offers the potential for reducing the volume of longitudinal reinforcement used in construction of one-way slabs. Slab deflections are relatively insensitive to steel strength and in regions where strength requirements control design of longitudinal reinforcement, the ratio of the required volume of HSR to Grade 60 reinforcement is approximately equal to the inverse of the ratio of the yield strengths of the reinforcement. The potential for HSR to reduce required reinforcement volumes is reduced by the need to meet ACI Code flexural crack width limits and in regions where ACI Code minimum reinforcement is required to control temperature and shrinkage cracking; the former can be addressed by the use of smaller reinforcing bars. Specific observations and conclusions regarding the use of HSR in design of one-ways slabs are as follows:

1. Deflection limits determine the required slab thickness. In regions of high flexural demand, strength requirements determined the reinforcement ratio. In regions of low demand, the reinforcement ratio is typically determined by the minimum area of reinforcement required by the ACI Code to control cracking due to temperature and shrinkage.
2. Deflections are essentially insensitive to steel strength as they are controlled primarily by the gross section moment of inertia. In regions of the system where flexural demand is low, the slab does not crack under service-level gravity loading, the effective stiffness is much greater than the cracked moment of inertia, and the gross-section moment of inertia controls deflections. In regions where the slab does crack under service-level gravity loading, the cracked moment of inertia is so much less than the gross-section moment of inertia for Grade 60 and HSR designs that the gross-section moment of inertia still controls deflection calculations.
3. Maximum flexural crack widths increase with increasing steel yield strength. Reducing bar size and bar spacing, while maintaining the overall reinforcement ratio, is the most economical approach to reducing flexural crack widths to meet ACI Code requirements. For Grade 60 reinforcement, design for strength and deflection requirements typically resulted is flexural crack widths that met Code limits. For f_y = 80 ksi designs, crack width requirement were met for most designs. For f_y = 120 ksi, most sections required redesign using with smaller bars (typically No. 3) and bar spacing to meet crack width limits.

Table 8 – Redesigned One-Way Slab Section

24' Span	**Previous Design – Section A**					**Redesign – Section A**					**Previous Design – Section B**					**Redesign – Section B**				
8"	**Rebar**	**ρ**	**$\varphi M_n/M_u$**	**I_{cr}/I_G**	**w_c [in]**	**Rebar**	**P**	**$\varphi M_n/M_u$**	**I_{cr}/I_G**	**w_c [in]**	**Rebar**	**ρ**	**$\varphi M_n/M_u$**	**I_{cr}/I_G**	**w_c [in]**	**Rebar**	**ρ**	**$\varphi M_n/M_u$**	**I_{cr}/I_G**	**w_c [in]**
60	**#4 @ 8"**	**0.31%**	**1.01**	**0.116**	**0.0150**	**#4 @ 8"**	**0.31%**	**1.01**	**0.116**	**0.0150**	**#4 @ 5"**	**0.50%**	**1.17**	**0.173**	**0.0092**	**#4 @ 5"**	**0.50%**	**1.17**	**0.173**	**0.0092**
80	#4 @ 10"	0.25%	1.08	0.096	0.0225	**#3 @ 6"**	**0.23%**	**1.00**	**0.091**	**0.0156**	**#4 @ 7"**	**0.36%**	**1.12**	**0.130**	**0.0161**	**#4 @ 7"**	**0.36%**	**1.12**	**0.130**	**0.0161**
120	#4 @ 14"	0.18%	1.16	0.071	0.0422	**#3 @ 6"**	**0.23%**	**1.49**	**0.091**	**0.0156**	#4 @ 11"	0.23%	1.07	0.088	0.0365	**#3 @ 5"**	**0.28%**	**1.30**	**0.106**	**0.0157**

27' Span	**Previous Design – Section A**					**Redesign – Section A**					**Previous Design – Section B**					**Redesign – Section B**				
10"	**Rebar**	**ρ**	**$\varphi M_n/M_u$**	**I_{cr}/I_G**	**w_c [in]**	**Rebar**	**ρ**	**$\varphi M_n/M_u$**	**I_{cr}/I_G**	**w_c [in]**	**Rebar**	**ρ**	**$\varphi M_n/M_u$**	**I_{cr}/I_G**	**w_c [in]**	**Rebar**	**ρ**	**$\varphi M_n/M_u$**	**I_{cr}/I_G**	**w_c [in]**
60	**#4 @ 7"**	**0.29%**	**1.04**	**0.121**	**0.0008**	**#4 @ 7"**	**0.29%**	**1.04**	**0.121**	**0.0008**	**#4 @ 5"**	**0.40%**	**1.06**	**0.161**	**0.0092**	**#4 @ 5"**	**0.40%**	**1.06**	**0.161**	**0.0092**
80	**#4 @ 9"**	**0.22%**	**1.08**	**0.097**	**0.0009**	**#4 @ 9"**	**0.22%**	**1.08**	**0.097**	**0.0009**	**#4 @ 7"**	**0.29%**	**1.01**	**0.121**	**0.0161**	**#4 @ 7"**	**0.29%**	**1.01**	**0.121**	**0.0161**
120	**#4 @ 11"**	**0.18%**	**1.32**	**0.081**	**0.0011**	**#4 @ 11"**	**0.18%**	**1.32**	**0.081**	**0.0011**	#4 @ 10"	0.20%	1.06	0.088	0.0365	**#3 @ 5"**	**0.22%**	**1.17**	**0.098**	**0.0175**

30' Span	**Previous Design – Section A**					**Redesign – Section A**					**Previous Design – Section B**					**Redesign – Section B**				
12"	**Rebar**	**ρ**	**$\varphi M_n/M_u$**	**I_{cr}/I_G**	**w_c [in]**	**Rebar**	**ρ**	**$\varphi M_n/M_u$**	**I_{cr}/I_G**	**w_c [in]**	**Rebar**	**ρ**	**$\varphi M_n/M_u$**	**I_{cr}/I_G**	**w_c [in]**	**Rebar**	**ρ**	**$\varphi M_n/M_u$**	**I_{cr}/I_G**	**w_c [in]**
60	**#4 @ 6"**	**0.28%**	**1.07**	**0.127**	**0.0007**	**#4 @ 6"**	**0.28%**	**1.07**	**0.127**	**0.0007**	**#4 @ 4"**	**0.42%**	**1.17**	**0.180**	**0.0081**	**#4 @ 4"**	**0.42%**	**1.17**	**0.180**	**0.0081**
80	**#4 @ 8"**	**0.21%**	**1.07**	**0.099**	**0.0009**	**#4 @ 8"**	**0.21%**	**1.07**	**0.099**	**0.0009**	**#4 @ 6"**	**0.28%**	**1.04**	**0.127**	**0.0156**	**#4 @ 6"**	**0.28%**	**1.04**	**0.127**	**0.0156**
120	**#4 @ 9"**	**0.19%**	**1.42**	**0.089**	**0.0010**	**#4 @ 9"**	**0.19%**	**1.42**	**0.089**	**0.0010**	#4 @ 9"	0.19%	1.04	0.089	0.0320	#3 @ 4"	0.23%	1.29	0.109	0.0140

Note: Bold font indicates a section design that meets ACI Code and standard practice deflection and crack width limits. All section designs meet ACI Code requirements and standard practice deflection limits. All redesigned sections meet ACI Code and standard practice deflection and crack width limits. For many sections, redesign was not required to meet crack width limits.

TWO-WAY SLABS

A series of two-way slab systems were designed to meet ACI Code requirements for gravity loading using RAM Concept software (http://www.bentley.com). A flat-plate system spanning four column lines in each direction (i.e. three bays by three bays) was assumed (Figure 2), and span lengths ranging from 24 ft. to 30 ft. were considered (Table 1). Designs were completed for Grade 60 and f_y = 80 ksi and 120 ksi mild reinforcement (Table 1). For each span length, section designs were completed for multiple slab thicknesses to meet flexural strength requirements. Load-history analysis in RAM Concept was used to compute slab deflections under sustained loading. Slab section designs meeting ACI Code and commonly employed deflection limits were identified. Flexural crack widths were computed for each section design using steel stress provided by RAM Concept and Eq. 3. Punching shear at column supports was not considered; in all cases, shear demand within the span was such that the concrete shear strength was adequate and no vertical reinforcement was required.

Table 9 provides details of the two-way slab designs, including the ratio of the weight of the longitudinal steel in the slab to the weight of the concrete in the slab, $\rho_{w,}$ and the maximum and minimum longitudinal reinforcement ratios, ρ_{max} and ρ_{min}, respectively. Figure 10 provides a graphical comparison of designs utilizing different strength steels for the case of 30 ft. spans and a 13 in. slab thickness.

FLEXURAL DESIGN FOR DESIGN-LEVEL DEMANDS

The RAM Concept software was used to design the two-way slab to meet flexural strength requirements. The RAM Concept model consisted of a three by three bay configuration with columns segments provided above and below the slab to provide appropriate stiffness in the column regions. The slab was meshed using an element size of 1/12 the span length. Gravity loads (Table 5) were applied with live load reduction implemented. Similar to the direct design approach, column and middle strips were identified in the model; the column strip extended five (5) times the thickness of the slab from the face of the column.

Table 9 and Figure 10 shows results for the two-way slab designs. For a 27 ft. span, 11 in. slab thickness and Grade 60 reinforcement, the longitudinal reinforcement ratio ranged from 0.40% (No. 5 @ 7 in.) to 0.18% (No. 4 @ 11 in.). For all designs, strength requirements determined the maximum reinforcement ratio and decreasing maximum reinforcement ratios were achieved with increasing steel yield strength. For all designs, the minimum reinforcement ratio was determined not by strength requirements but by Code requirements intended to control cracking due to temperature and shrinkage (ACI Code Section 7.12) or by Code minimum spacing requirements (Section 13.3.2). Code minimum reinforcement requirements controlled design for much of the slab; thus, the reduction in total steel weight, ρ_w, achieved by using HSR was modest (approximately 20%) for designs meeting deflection limits.

EVALUATION OF PERFORMANCE UNDER SERVICE-LEVEL LOADING

The performance of the strength-based designs under service-level loading was assessed on the basis of i) the maximum deflection of the slab and ii) the width of flexural cracks. RAM Concept was used to determine deflections under sustained loading, with load-history calculation options calibrated to provide results consistent with other commercial software. Maximum deflections were observed at approximately mid-span of the exterior bays. Maximum deflections were moderately insensitive to the reinforcement ratio of the section, with an increase in steel yield strength and reduction in steel reinforcement ratio resulting in at most an 18% increase in maximum deflection. Flexural cracks widths were computed using steel stresses provided by RAM Concept software and Eq. 3. As with one-way slabs, maximum flexural crack widths increased with increasing steel strength; most designs utilizing f_y = 80 ksi and f_y = 120 ksi reinforcement did not meet ACI Code intended limits on flexural crack widths. As for one-way slabs, it is expected that flexural crack widths could be reduced to meet Code intended limits by redesigning reinforcement to employ smaller, more closely spaced reinforcing bars.

Table 9 - Two-Way Slab Results

24’ Span		ρ_w	Max Rebar	ρ_{max}	Min Rebar	ρ_{min}	Δ_{max} [in]	w_c [in]
7”	60	3.93%	#5 @ 6”	0.74%	#5 @ 13” [2]	0.34%	-1.02	0.0112[1]
	80	3.83%	#5 @ 8”	0.55%	#5 @ 13” [2]	0.34%	-1.08	0.0191
	120	3.80%	#5 @ 13”	0.34%	#5 @ 13” [2]	0.34%	-1.20	0.0487
8”	60	3.06%	#5 @ 7”	0.55%	#5 @ 16” [2]	0.24%	-0.80	0.0135[2]
	80	2.98%	#5 @ 9”	0.43%	#5 @ 16” [2]	0.24%	-0.84	0.0217
	120	2.95%	#5 @ 14”	0.28%	#5 @ 16” [2]	0.24%	-0.87	0.0509
9”	**60**	**2.34%**	**#5 @ 8”**	**0.43%**	**#4 @ 12” [2]**	**0.19%**	**-0.64[1]**	**0.0156[1]**
	80	1.92%	#5 @ 10”	0.34%	#5 @ 18” [2]	0.19%	-0.65[1]	0.0244
	120	1.88%	#5 @ 16”	0.22%	#4 @ 15” [2]	0.15%	-0.69[1]	0.0607

27’ Span		ρ_w	Max Rebar	ρ_{max}	Min Rebar	ρ_{min}	Δ_{max} [in]	w_{c_max} [in]
9”	60	2.40%	#5 @ 6”	0.57%	#5 @ 18” [2]	0.19%	-1.00	0.0125[1]
	80	1.94%	#5 @ 8”	0.43%	#4 @ 15” [2]	0.15%	-1.04	0.0214
	120	1.80%	#5 @ 12”	0.29%	#4 @ 15” [2]	0.15%	-1.11	0.0466
10”	60	2.22%	#5 @ 6”	0.52%	#5 @ 17” [2]	0.18%	-0.85	0.0116[1]
	80	1.84%	#5 @ 8”	0.39%	#4@14” [2]	0.14%	-0.85	0.0199
	120	1.80%	#5 @ 13”	0.24%	#4 @ 14” [2]	0.14%	-0.89	0.0508
11”	**60**	**2.17%**	**#5 @ 7”**	**0.40%**	**#4@ 10” [2]**	**0.18%**	**-0.71[1]**	**0.0144[1]**
	80	1.75%	#5 @ 9”	0.31%	#4 @ 12” [2]	0.15%	-0.71[1]	0.0233
	120	1.64%	#4 @ 9”	0.20%	#4 @ 12” [2]	0.15%	-0.74[1]	0.0352

30’ Span		ρ_w	Max Rebar	ρ_{max}	Min Rebar	ρ_{min}	Δ_{max} [in]	w_{c_max} [in]
11”	60	2.35%	#5 @ 5”	0.56%	#5 @ 15” [2]	0.19%	-1.06	0.0101[1]
	80	1.95%	#5 @ 7”	0.40%	#5 @ 18” [2]	0.16%	-1.08	0.0188
	120	1.58%	#4 @ 6”	0.30%	#4 @ 12” [2]	0.15%	-1.12	0.0211
12”	60	2.21%	#5 @ 5”	0.52%	#5 @ 14” [2]	0.18%	-0.92	0.0096[1]
	80	1.70%	#5 @ 7”	0.37%	#4@11” [2]	0.15%	-0.94	0.0178[1]
	120	1.60%	#4 @ 7”	0.24%	#4 @ 11” [2]	0.15%	-0.95	0.0268
13”	**60**	**2.11%**	**#5 @ 5”**	**0.48%**	**#5@ 13” [2]**	**0.18%**	**-0.75[1]**	**0.0091[1]**
	80	1.76%	#5 @ 7”	0.34%	#4 @ 10” [2]	0.15%	-0.76	0.0170[1]
	120	1.71%	#4 @ 7”	0.22%	#4 @ 10” [2]	0.15%	-0.80	0.0256

Note: Bold font indicates a design that meets ACI Code and standard practice deflection and crack width limits. 1. Meets ACI Code and standard practice deflection or crack width limits. 2. Design is controlled by ACI Code Section 7.12 or by ACI Code Section 13.3.2.

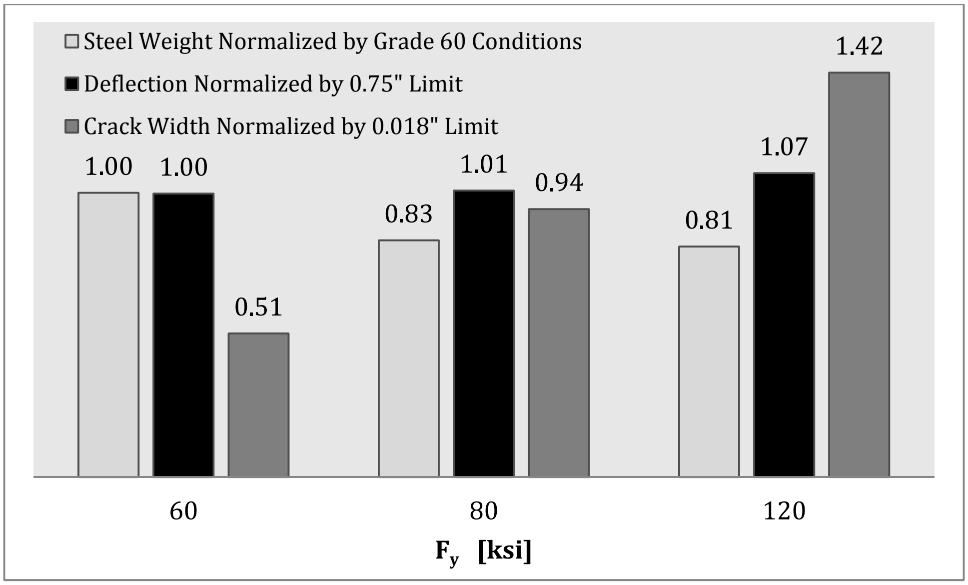


Figure 10 - Two-Way Slab Example – 30 ft. Span Length, 13 in. Slab Thickness

CONCLUSIONS REGARDING DESIGN OF TWO-WAY SLABS USING HSR

The use of HSR offers potential for reducing the volume of longitudinal reinforcement used in construction of two-way slabs. Slab deflections are relatively insensitive to steel strength. For slab thicknesses that meet deflection limits, Code minimum reinforcement requirements control design of longitudinal reinforcement for a significant portion of the two-way slab system. Thus, the extent to which the required volume of longitudinal reinforcement can be reduced using HSR is limited. The potential for HSR to reduce required reinforcement volumes and, thus, construction cost is limited also by the need to meet ACI Code flexural crack width limits. Designs utilizing No. 4 and No. 5 HSR bars did not meet flexural crack width limits; these limits likely can be met by using smaller reinforcing bars. Specific observations and conclusions regarding the use of HSR in design of two-ways slabs are as follows:

1. As with one-way slabs, deflection limits determine slab thickness. Deflections are relatively insensitive to the slab reinforcement ratios. For 24 ft. and 27 ft. span lengths, use of a higher strength steel did not result in a section failing to meet the deflection limits.
2. Code required minimum reinforcement controls design in many regions of the system with the result that for designs meeting deflection requirements only a modest reduction in total steel weight (approximately 20%) is realized by using HSR.
3. Flexural crack widths increase with increasing steel strength. Most designs utilizing HSR do not meet Code intended limits on flexural crack widths. This could be mitigated without increasing steel volume by using smaller, more closely spaced reinforcing bars. However, the use of smaller bars could be expected to increase congestion and cost of construction.

BEAMS

A series of beams were designed to meet ACI Code requirements under gravity loading; the design process for beams was similar to that employed for one-way slabs. A continuous three-span system was assumed (Figure 3), and span lengths ranging from 24 ft. to 30 ft. were considered. Beams were assumed to be part of a slab-beam system, such that a T-beam section geometry was employed (Figure 4). Designs were completed for Grade 60 and f_y = 80 ksi and 120 ksi mild reinforcement (Table 2). For each span length, sections designs were completed to meet strength requirements for multiple beam depths. The ACI Code required minimum longitudinal reinforcement ratio for beams (ACI Code Section 10.5) was employed as necessary. Deflections under service-level loading were computed using elastic analysis with effective stiffnesses at each section defined per Eq. 1 and average effective stiffnesses for each span computed as previously discussed. Long-term deflections under sustained loads were determined using Eq. 2. For each section design, flexural crack widths were computed using Eq. 3. Beam section designs meeting ACI Code and commonly employed deflection limits as well as flexural crack width limits were identified. In all cases, shear demands were such that only minimum vertical reinforcement was required.

Table 10 provides details of the beam flexural designs; data are presented as for one-way slabs. Figure 11 provides a graphical comparison of beam designs utilizing different strength steels for the case of 30 ft. spans and a 26 in. deep beam. Table 11 provides shear design details for an exterior 27 ft. span.

STRENGTH DESIGN FOR DESIGN-LEVEL DEMANDS

Elastic analysis and the RISA-3D structural analysis software (http://www.risatech.com/) were used to determine moment and shear demands for design. Previously described gravity loads were used (Table 5). Given the symmetry of the three-span system, three different slab sections were designed (Figure 3): Sections A and B located, respectively, at the points of maximum positive and negative moment in the exterior spans and Section C located at the point of maximum positive moment in the interior span. As with slab design, compression reinforcement was ignored in the section design. For design of Section B in which the T-beam slab carries tension, minimum slab reinforcement (Section 7.12.2.1) was assumed.

Table 10 provides details of the beam designs, and Figure 11 provides a graphical presentation of design details for a particular beam configuration (30 ft. span length and 26 in. deep beam). For 27 ft. spans, a 26 in. deep beam, and Grade 60 reinforcement, the longitudinal reinforcement ratio ranged from 0.17% (4 No. 4) to 0.56% (6 No. 6). As with the one-way slab designs, in regions of high flexural demand, the ratio of the required areas of HSR to Grade 60 reinforcement was approximately equal to the inverse of the ratio of the yield strengths of the steel. The area of HSR was increased beyond this at sections with low flexural demands to meet Code requirements for minimum reinforcement to ensure that the nominal flexural strength exceeds cracking strength (Section 10.5.1). Sections for which minimum reinforcement requirements controlled design are noted Table 10. For beams, strength requirements controlled design for a relatively larger portion of the system and the use of HSR results in a greater reduction in total steel volume than was observed for slabs (Figure 11).

Shear demand on beams was minimal, and design of transverse reinforcement was controlled by the maximum spacing limits specified in ACI Code Section 11.4.5, specifically the requirement that transverse reinforcement be spaced at less $0.5d$, where d is the depth of the longitudinal reinforcement. Table 11 provides shear design details for an exterior span of length 27 ft. Since maximum spacing limits controlled shear design, steel strength had no impact on design.

Table 10- Beam Results

27' Span		Section A				Section B				Section C				Exterior Span				Interior Span			
		Rebar	ρ	$\varphi M_n/M_u$	I_{cr}/I_G	Rebar	ρ	$\varphi M_n/M_u$	I_{cr}/I_G	Rebar	ρ	$\varphi M_n/M_u$	I_{cr}/I_G	I_D/I_G	I_{D+L}/I_G	Δ_{TOT} [in]	w_c [in]	I_D/I_G	I_{D+L}/I_G	Δ_{TOT} [in]	w_c [in]
22"	60	4 No. 8	0.80%	1.12	0.225	4 No. 7	0.61%	1.01	0.243	3 No. 5 [2]	0.23%	1.53	0.074	0.52	0.40	-0.83	0.0107[1]	1.00	0.81	0.11[1]	0.0096[1]
	80	5 No. 6	0.56%	1.04	0.164	5 No. 5	0.39%	1.03	0.212	4 No. 4 [2]	0.20%	1.76	0.065	0.48	0.36	-0.91	0.0143[1]	1.00	0.80	0.12[1]	0.0108[1]
	120	5 No. 5	0.39%	1.11	0.120	4 No. 4	0.20%	1.12	0.181	3 No. 4 [2]	0.15%	1.98	0.049	0.45	0.32	-0.98	0.0195	1.00	0.80	0.13[1]	0.0128[1]
24"	**60**	**5 No. 7**	**0.69%**	**1.16**	**0.204**	**5 No. 6**	**0.51%**	**1.05**	**0.227**	**4 No. 4 [2]**	**0.19%**	**1.44**	**0.061**	**0.58**	**0.50**	**-0.56[1]**	**0.0100[1]**	**1.00**	**0.99**	**0.07[1]**	**0.0088[1]**
	80	**5 No. 6**	**0.51%**	**1.14**	**0.155**	**4 No. 5**	**0.29%**	**1.01**	**0.191**	**4 No. 4 [2]**	**0.19%**	**1.92**	**0.061**	**0.55**	**0.46**	**-0.60[1]**	**0.0131[1]**	**1.00**	**0.99**	**0.08[1]**	**0.0107[1]**
	120	**5 No. 5**	**0.36%**	**1.37**	**0.113**	**4 No. 4**	**0.19%**	**1.23**	**0.173**	**2 No. 4 [2]**	**0.09%**	**1.44**	**0.031**	**0.52**	**0.43**	**-0.63[1]**	**0.0179[1]**	**1.00**	**0.99**	**0.08[1]**	**0.0117[1]**
26"	**60**	**6 No. 6**	**0.56%**	**1.11**	**0.173**	**6 No. 5**	**0.40%**	**1.04**	**0.206**	**4 No. 4 [2]**	**0.17%**	**1.56**	**0.058**	**0.68**	**0.55**	**-0.40[1]**	**0.0101[1]**	**1.00**	**1.00**	**0.05[1]**	**0.0016[1]**
	80	**6 No. 5**	**0.40%**	**1.05**	**0.126**	**4 No. 5**	**0.26%**	**1.10**	**0.182**	**4 No. 4 [2]**	**0.17%**	**2.07**	**0.058**	**0.65**	**0.51**	**-0.42[1]**	**0.0138[1]**	**1.00**	**1.00**	**0.05[1]**	**0.0016[1]**
	120	4 No. 5	0.26%	1.05	0.087	4 No. 4	0.17%	1.33	0.165	2 No. 4 [2]	0.09%	1.56	0.030	0.63	0.49	-0.43[1]	0.0206	1.00	1.00	0.05[1]	0.0016[1]

30' Span		Section A				Section B				Section C				Exterior Span				Interior Span			
		Rebar	ρ	$\varphi M_n/M_u$	I_{cr}/I_G	Rebar	ρ	$\varphi M_n/M_u$	I_{cr}/I_G	Rebar	ρ	$\varphi M_n/M_u$	I_{cr}/I_G	I_D/I_G	I_{D+L}/I_G	Δ_{TOT} [in]	w_c [in]	I_D/I_G	I_{D+L}/I_G	Δ_{TOT} [in]	w_c [in]
26"	**60**	**5 No. 7**	**0.64%**	**1.10**	**0.189**	**5 No. 6**	**0.47%**	**1.03**	**0.220**	**4 No. 4 [2]**	**0.17%**	**1.30**	**0.056**	**0.54**	**0.47**	**-0.70[1]**	**0.0113[1]**	**1.00**	**1.00**	**0.08[1]**	**0.0020[1]**
	80	**5 No. 6**	**0.47%**	**1.07**	**0.143**	**5 No. 5**	**0.33%**	**1.11**	**0.198**	**4 No. 4 [2]**	**0.17%**	**1.73**	**0.056**	**0.51**	**0.44**	**-0.74[1]**	**0.0148[1]**	**1.00**	**1.00**	**0.09[1]**	**0.0020[1]**
	120	5 No. 5	0.33%	1.14	0.104	3 No. 4	0.13%	1.11	0.162	3 No. 4 [2]	0.13%	1.95	0.042	0.49	0.41	-0.78	0.0202	1.00	1.00	0.10[1]	0.0024[1]
28"	**60**	**5 No. 7**	**0.60%**	**1.11**	**0.179**	**4 No. 7**	**0.48%**	**1.10**	**0.217**	**3 No. 5 [2]**	**0.18%**	**1.53**	**0.061**	**0.63**	**0.52**	**-0.51[1]**	**0.0105[1]**	**1.00**	**1.00**	**0.06[1]**	**0.0018[1]**
	80	**5 No. 6**	**0.44%**	**1.09**	**0.136**	**4 No. 5**	**0.25%**	**1.02**	**0.178**	**4 No. 4 [2]**	**0.16%**	**1.76**	**0.053**	**0.60**	**0.49**	**-0.53[1]**	**0.0138[1]**	**1.00**	**1.00**	**0.07[1]**	**0.0017[1]**
	120	5 No. 5	0.31%	1.16	0.098	3 No. 4	0.12%	1.14	0.155	3 No. 4 [2]	0.12%	1.98	0.040	0.58	0.47	-0.55[1]	0.0188	1.00	1.00	0.07[1]	0.0021[1]
30"	**60**	**6 No. 6**	**0.49%**	**1.05**	**0.152**	**5 No. 6**	**0.41%**	**1.12**	**0.203**	**4 No. 4 [2]**	**0.15%**	**1.40**	**0.050**	**0.73**	**0.58**	**-0.37[1]**	**0.0108[1]**	**1.00**	**1.00**	**0.04[1]**	**0.0016[1]**
	80	**5 No. 6**	**0.41%**	**1.17**	**0.129**	**4 No. 5**	**0.23%**	**1.09**	**0.171**	**4 No. 4 [2]**	**0.15%**	**1.87**	**0.050**	**0.72**	**0.57**	**-0.38[1]**	**0.0129[1]**	**1.00**	**1.00**	**0.04[1]**	**0.0015[1]**
	120	**5 No. 5**	**0.29%**	**1.23**	**0.093**	**3 No. 4**	**0.11%**	**1.22**	**0.149**	**3 No. 4 [2]**	**0.11%**	**2.10**	**0.038**	**0.70**	**0.55**	**-0.39[1]**	**0.0176[1]**	**1.00**	**1.00**	**0.04[1]**	**0.0019[1]**

33' Span		Section A				Section B				Section C				Exterior Span				Interior Span			
		Rebar	ρ	$\varphi M_n/M_u$	I_{cr}/I_G	Rebar	ρ	$\varphi M_n/M_u$	I_{cr}/I_G	Rebar	ρ	$\varphi M_n/M_u$	I_{cr}/I_G	I_D/I_G	I_{D+L}/I_G	Δ_{TOT} [in]	w_c [in]	I_D/I_G	I_{D+L}/I_G	Δ_{TOT} [in]	w_c [in]
28"	60	5 No. 8	0.78%	1.23	0.224	5 No. 7	0.60%	1.09	0.237	4 No. 4 [2]	0.16%	1.07	0.052	0.54	0.49	-0.80	0.0099[1]	1.00	1.00	0.09[1]	0.0022[1]
	80	5 No. 7	0.60%	1.24	0.176	4 No. 6	0.35%	1.05	0.199	4 No. 4 [2]	0.16%	1.43	0.052	0.51	0.46	-0.85	0.0125[1]	1.00	1.00	0.10[1]	0.0021[1]
	120	4 No. 6	0.35%	1.10	0.108	4 No. 4	0.16%	1.10	0.168	3 No. 4 [2]	0.12%	1.61	0.039	0.46	0.41	-0.93	0.0205	1.00	1.00	0.11[1]	0.0020[1]
30"	**60**	**7 No. 6**	**0.57%**	**1.03**	**0.172**	**6 No. 6**	**0.49%**	**1.08**	**0.219**	**3 No. 5 [2]**	**0.17%**	**1.33**	**0.056**	**0.58**	**0.50**	**-0.63[1]**	**0.0111[1]**	**1.00**	**1.00**	**0.07[1]**	**0.0019[1]**
	80	**6 No. 6**	**0.49%**	**1.17**	**0.149**	**5 No. 5**	**0.29%**	**1.06**	**0.186**	**4 No. 4 [2]**	**0.15%**	**1.52**	**0.049**	**0.57**	**0.48**	**-0.65[1]**	**0.0129[1]**	**1.00**	**1.00**	**0.07[1]**	**0.0018[1]**
	120	4 No. 6	0.33%	1.17	0.102	4 No. 4	0.15%	1.18	0.161	3 No. 4 [2]	0.11%	1.71	0.037	0.53	0.45	-0.68[1]	0.0192	1.00	1.00	0.08[1]	0.0018[1]
32"	**60**	**5 No. 7**	**0.52%**	**1.06**	**0.159**	**5 No. 6**	**0.38%**	**1.03**	**0.198**	**5 No. 4 [2]**	**0.17%**	**1.51**	**0.057**	**0.67**	**0.56**	**-0.47[1]**	**0.0110[1]**	**1.00**	**1.00**	**0.05[1]**	**0.0017[1]**
	80	**5 No. 6**	**0.38%**	**1.04**	**0.120**	**5 No. 5**	**0.27%**	**1.13**	**0.179**	**4 No. 4 [2]**	**0.14%**	**1.61**	**0.046**	**0.65**	**0.53**	**-0.49[1]**	**0.0145[1]**	**1.00**	**1.00**	**0.05[1]**	**0.0016[1]**
	120	4 No. 6	0.31%	1.25	0.097	2 No. 4	0.07%	1.05	0.141	3 No. 4 [2]	0.10%	1.82	0.035	0.63	0.52	-0.50[1]	0.0181	1.00	1.00	0.05[1]	0.0035[1]

Notes: Bold font indicates a design that meets ACI Code and standard practice deflection and crack width limits. 1. Meets ACI Code and standard practice deflection or crack width limits. 2. Design was controlled by ACI Code Section 10.5, which specifies a minimum longitudinal reinforcement ratio to ensure that nominal flexural strength exceeds cracking strength.

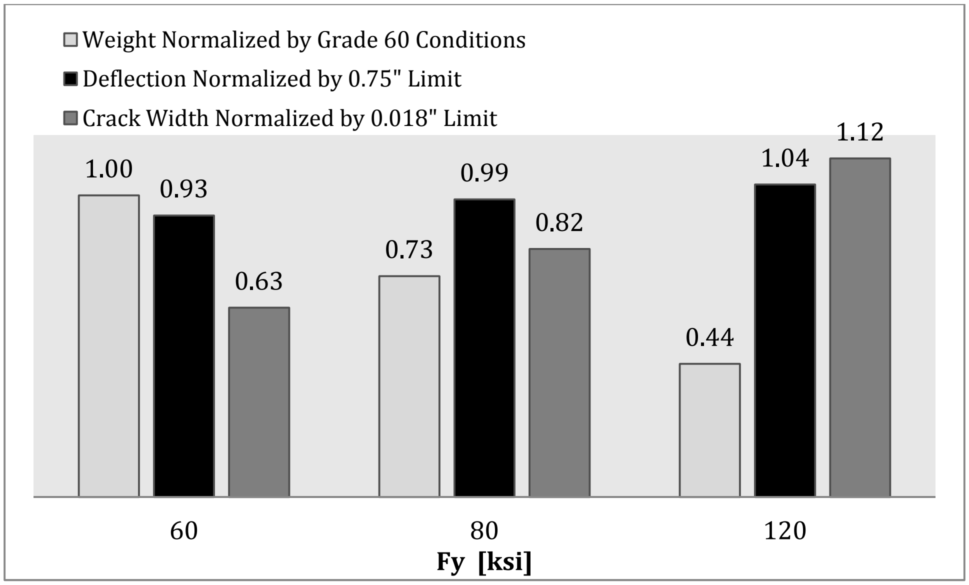


Figure 11 - Beam Example – 30 ft. Span Length, 26 in. Beam Depth

Table 11- Shear Design of Beams Sample

27' Exterior Span									
		Outermost Section of Span				**Innermost Section of Span**			
		s [in]	**Bar No.**	**A_v [in²]**	**$\varphi V_n/V_u$**	**s [in]**	**Bar No.**	**A_v [in²]**	**$\varphi V_n/V_u$**
22"	**60**	9	4	0.4	1.64	9	4	0.4	1.10
	80	9	4	0.4	1.92	9	4	0.4	1.29
	120	9	4	0.4	2.48	9	4	0.4	1.66
24"	**60**	10	4	0.4	1.70	10	4	0.4	1.14
	80	10	4	0.4	1.98	10	4	0.4	1.32
	120	10	4	0.4	2.53	10	4	0.4	1.69
26"	**60**	11	4	0.4	1.77	11	4	0.4	1.18
	80	11	4	0.4	2.04	11	4	0.4	1.36
	120	11	4	0.4	2.57	11	4	0.4	1.72

EVALUATION OF PERFORMANCE UNDER SERVICE-LEVEL LOADING

The performance of the beams was evaluated on the basis of i) the maximum deflection of the beam and ii) the width of flexural cracks. The maximum deflection in the three-span beam system occurred in the vicinity of the maximum positive moment in the exterior span (Section A in Figure 3); the interior span experienced upwards deflections of smaller magnitude. As was the case for one-way slabs, beam deflections were relatively insensitive to steel strength and reinforcement ratio. This was particularly true for the shallower beam designs; for all beam designs, deflections increased by a maximum of 18% with increasing steel yield strength. As for the one-way slabs, this results from the fact that much of the beam remains uncracked under service-level loads and gross section properties control deflections.

Table 10 lists maximum flexural crack widths developed under service-level loading. Maximum crack widths were observed for the positive moment region of the exterior span (Section A in Figure 3). Maximum crack width increases with increasing steel strength. In all cases, sections designed to meet strength requirements using Grade 60 and f_y = 80ksi reinforcement meet intended ACI Code crack width limits; designs utilizing f_y = 120ksi exceed intended ACI Code crack width limits by at most 0.0026 in., which is 14% of the limit.

CONCLUSIONS REGARDING DESIGN OF BEAMS USING HSR

The use of HSR offers the potential for reducing the volume of longitudinal reinforcement used in concrete beams. Beam deflections are relatively insensitive to steel strength and in regions where strength requirements control design of longitudinal reinforcement, the ratio of the required volume of HSR to Grade 60 reinforcement is approximately equal to the inverse of the ratio of the yield strengths of the reinforcement. The potential for HSR to reduce required reinforcement volumes is reduced by the need to meet ACI Code flexural crack width limits and in regions where ACI Code minimum reinforcement is required to ensure nominal flexural strength exceeds cracking strength; the former can be addressed by the use of smaller reinforcing bars. Specific observations and conclusions regarding the use of HSR in design of beams are as follows:

1. Beam depth is controlled by deflections under service-level loading. In most regions of the system, longitudinal reinforcement is controlled by strength requirements for design-level loading. In regions of low flexural demand (interior span, Section C in Figure 3), the design is controlled by the minimum reinforcement requirements.
2. Deflections are relatively insensitive to steel yield strength, with maximum deflections increasing at most by 18% when steel yield strength was increased from f_y = 60ksi to f_y = 120ksi.
3. In most regions of the system, longitudinal reinforcement is controlled by strength requirements for design-level loading, with the result that the use of HSR significantly reduces steel volume.
4. Maximum flexural crack widths are dependent on the layout of beam reinforcement and beam depth; design to meet crack width limits requires the use of a greater number of smaller bars or increased beam depth.
5. Shear design is controlled by ACI Code maximum spacing requirements for transverse reinforcement and is therefore not affected by steel strength.

GRAVITY COLUMNS

Three rectangular reference column geometries were designed using Grade 60 reinforcement and the ACI Code minimum longitudinal reinforcement ratio of 1%. P-M interaction curves, defining the nominal flexural strength of the column under varying axial load, were generated for these reference column geometries for concrete compressive strengths of 6,000 psi and 10,000 psi. Columns were then redesigned using HSR to achieve the same factored flexural strength as the Grade 60 designs at axial loads ranging from 10% to 30% of factored axial strength. P-M interaction curves were generated for the HSR designs. Confining and shear reinforcement were designed for each column configuration, grades of reinforcement, and concrete compressive strengths. Confining and shear reinforcement were designed per ACI Code Section 21.13, which applies to gravity columns in regions of high seismicity.

LONGITUDINAL REINFORCEMENT

Figure 5 shows the three column cross-section geometries considered in the study. Columns are 24 in. wide and range in depth from 24 in. to 36 in; longitudinal reinforcement consists of 8, 10 and 12 bars depending on the column geometry. The Grade 60 reference columns were redesigned using HSR with f_y = 80 ksi, 100 ksi and 120 ksi. For all steel grades, columns constructed using f_c' = 6000 psi and 10000 psi were considered.

The software spColumn (http://www.structurepoint.org) was used to generate P-M interaction curves for all column configurations. Appropriate strength reduction factors for tension-controlled (flexural members) and compression-controlled (compression members) response, φ = 0.9 and φ = 0.65 respectively, were considered in generating the curves. For a particular cross-section geometry and concrete compressive strength, the P-M curve was generated using Grade 60 reinforcement and a 1% reinforcement ratio. The longitudinal reinforcement was then redesigned using HSR to achieve the same factored flexural strength at an axial load of 10% of the factored axial strength, and a new P-M curve was generated for this cross-section design. The process of redesigning the section and generating new P-M curves was then repeated to achieve comparable flexural strength at axial loads equal to 20% and 30% of the factored axial strength of the Grade 60 design. Table 12 presents longitudinal reinforcement ratios for the Column A and C designs. Figures 12 and 13 shows P-M interaction curves for the Column A cross section and all steel strengths, where all section designs have approximately the same flexural strength at an axial load of 10% of the factored axial strength (Figure 12) and 30% of the factored axial strength (Figure 13). Figure 14 shows P-M interaction curves for the Column A cross section and f_y = 120 ksi, where sections designs have approximately the same flexural strength as the reference column (Grade 60 reinforcement) at axial loads of 10%, 20% and 30% of the factored axial strength.

Table 12 - Column Longitudinal Steel Results

Column A	**6000 psi**						**10000 psi**					
	$A_{s\_10\%}$ [in²]	$\rho_{\_10\%}$	$A_{s\_20\%}$ [in²]	$\rho_{\_20\%}$	$A_{s\_30\%}$ [in²]	$\rho_{\_30\%}$	$A_{s\_10\%}$ [in²]	$\rho_{\_10\%}$	$A_{s\_20\%}$ [in²]	$\rho_{\_20\%}$	$A_{s\_30\%}$ [in²]	$\rho_{\_30\%}$
60	6.32	1.10%	6.32	1.10%	6.32	1.10%	6.32	1.10%	6.32	1.10%	6.32	1.10%
80	4.80	0.83%	4.80	0.83%	4.80	0.83%	4.80	0.83%	4.80	0.83%	4.80	0.83%
100	3.84	0.67%	3.84	0.67%	4.16	0.72%	3.84	0.67%	3.84	0.67%	8.00	1.39%
120	3.26	0.57%	3.52	0.61%	8.00	1.39%	3.26	0.57%	3.52	0.61%	9.08	1.58%

Column C	**6000 psi**						**10000 psi**					
	$A_{s\_10\%}$ [in²]	$\rho_{\_10\%}$	$A_{s\_20\%}$ [in²]	$\rho_{\_20\%}$	$A_{s\_30\%}$ [in²]	$\rho_{\_30\%}$	$A_{s\_10\%}$ [in²]	$\rho_{\_10\%}$	$A_{s\_20\%}$ [in²]	$\rho_{\_20\%}$	$A_{s\_30\%}$ [in²]	$\rho_{\_30\%}$
60	10.00	1.16%	10.00	1.16%	10.00	1.16%	10.00	1.16%	10.00	1.16%	10.00	1.16%
80	7.90	0.91%	7.90	0.91%	7.90	0.91%	7.14	0.83%	7.90	0.91%	7.90	0.91%
100	6.00	0.69%	6.38	0.74%	6.38	0.74%	6.00	0.69%	6.38	0.74%	6.38	0.74%
120	5.04	0.58%	5.04	0.58%	11.62	1.34%	5.04	0.58%	5.04	0.58%	13.86	1.60%

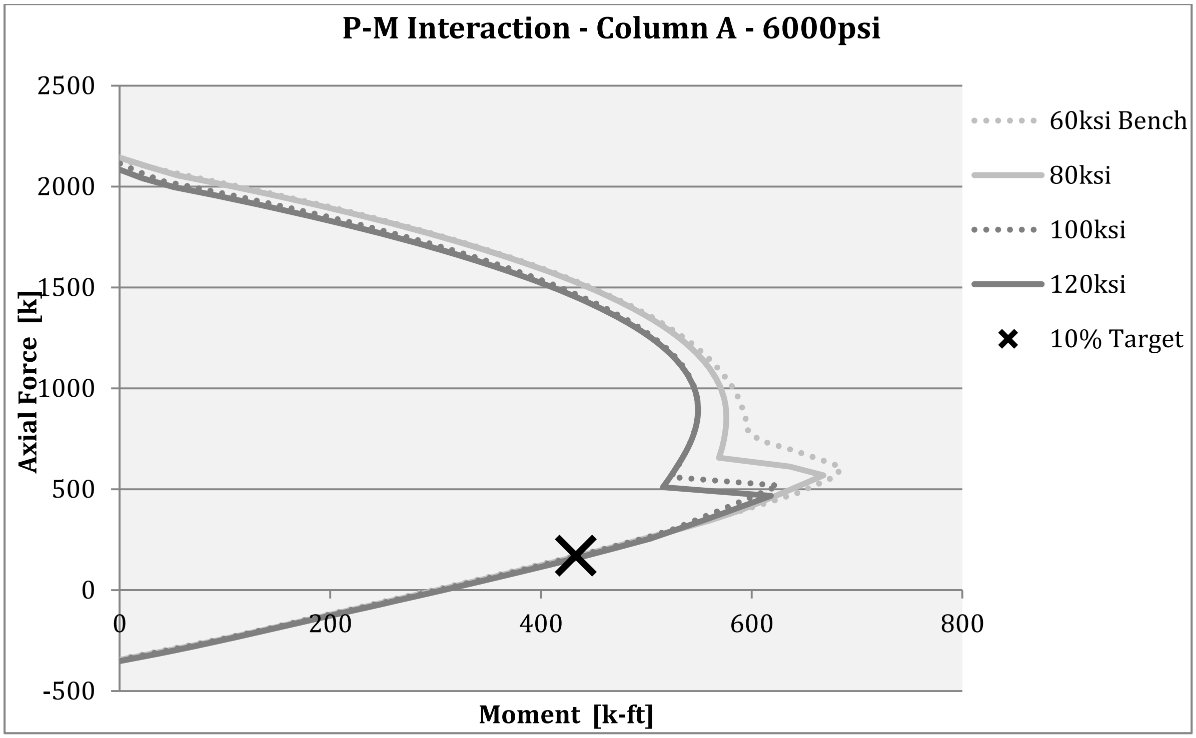


Figure 12- P-M Interaction Curves for Column A (Fig. 5) with f'_c = 6,000 psi and Varying f_y. All designs have the same flexural strength at axial load ratio of 10%.

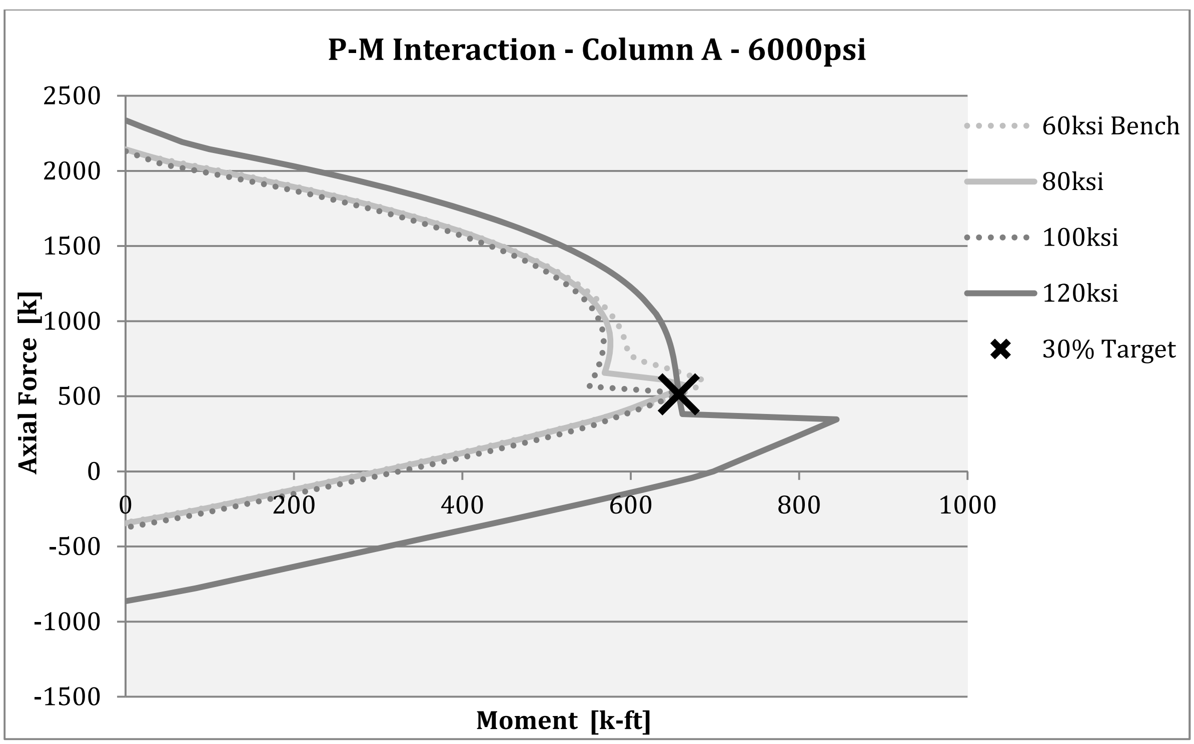


Figure 13 **- P-M Interaction Curves for Column A (Fig. 5) with f'_c = 6,000 psi and Varying f_y. All designs have the same flexural strength at axial load ratio of 30%.**

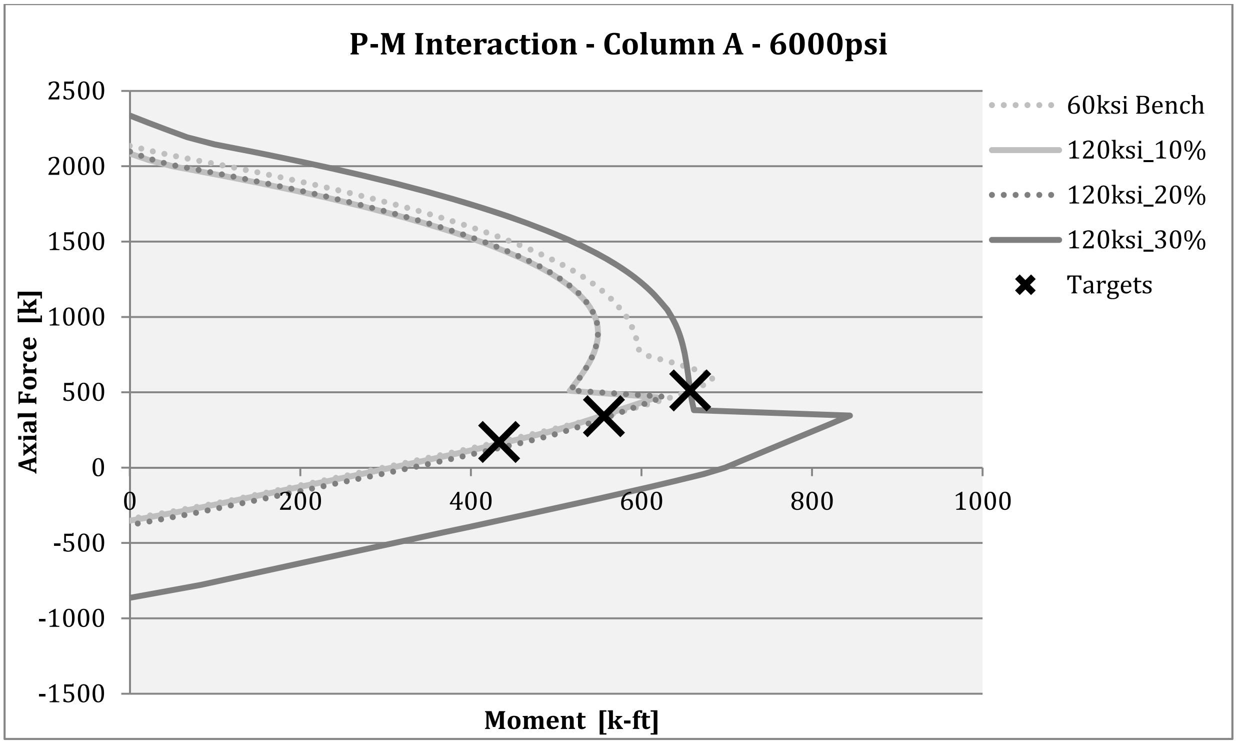


Figure 14 - P-M Interaction Curves for Column A (Fig. 5) with f'$_c$ = 6,000 psi and f$_y$ =120 ksi. Designs have the same flexural strength as the 60 ksi reference design for axial load ratios of 10%, 20% and 30%.

Observations

For gravity columns with low axial load demands exhibiting tension-controlled response (i.e. flexural members), the use of HSR reduces the volume of longitudinal reinforcement required to meet flexural strength requirements and the ratio of required HSR to Grade 60 longitudinal reinforcement is approximately equal to the inverse of the ratio of the steel yield strengths. Design data (Table 12) for column configurations with axial loads of 10% and 20% of factored axial strength show this trend. In practice, for gravity columns with high axial load demands exhibiting compression-controlled response (i.e. compression members), response is determined by column geometry and concrete strength and the Code required minimum reinforcement ratio determined design of the longitudinal reinforcement. For these columns, designs could be expected to be insensitive to steel strength. For the current study, designs utilizing HSR were not developed to match the factored flexural strength of Grade 60 designs for high axial load demands. In the vicinity of the transition from tension- to compression-controlled response, factored flexural strength is highly sensitive to the response mode (tension- versus compression-controlled) and the resulting strength reduction factor. For the column configuration considered in this study, the transition from tension- to compression-controlled response occurred in the vicinity of 30% of the factored axial strength. The data in Figures 12-14 show that the use of HSR results in columns that exhibit compression-controlled response at slightly lower axial loads than columns designed using Grade 60 reinforcement. Thus, gravity columns designed using HSR for the case of an axial load demand of 30% of factored axial strength required a significantly *larger* volume of longitudinal reinforcement to achieve the same *factored* flexural strength as a column designed using Grade 60 reinforcement. Design data in Table 12 for column configurations with axial loads of 30% of factored axial strength show this trend. As this is an artifact of the discrete transition from tension- to compression-controlled response, it is unlikely that this would control column design using HSR.

ACI Code Section 10.9.1 specifies a minimum longitudinal reinforcement ratio of 1% for compression members. This is intended to ensure that under sustained compressive load, load transfer from concrete to steel due to concrete creep does not result in yielding of the reinforcing steel as well as to ensure that compression members have nominal flexural strength. Original minimum longitudinal reinforcement limits were 0.5% and 1.0% for compression members with tied and spiral transverse reinforcement, respectively (ACI Com. 105, 1933); the 1% minimum was introduced in 1936. Given the higher yield strength of HSR, similar safety against yielding under sustained loads could likely be achieved using a lower reinforcement ratio than required for Grade 60 reinforcement. This limit controls design of gravity columns with higher axial loads in this study.

CONFINING REINFORCEMENT

For buildings in regions of high seismicity, rectangular compression-controlled gravity columns must meet the requirements of ACI Code Sections 7.10.5 and 21.13; the requirements of Section 21.13 controlled the design in all cases. These ACI Code Sections specify the configuration, spacing and volume of transverse reinforcement required in regions of length l_o at column ends where flexural yielding may occur under earthquake loading. Transverse reinforcement in these regions is intended to confine core concrete and restrain buckling of longitudinal reinforcement following spalling of cover concrete under earthquake loading. For this study, flexural demands under earthquake loading were not computed; in this case, ACI Code Section 21.13 specifies that confinement must meet the more stringent requirements of Sections 21.6.4. Specifically, transverse reinforcement must have a center-to-center spacing, s_o, of between 6 in. and 4 in. but not greater than

$$s_o = 4 + \left(\frac{14 - h_x}{3}\right) \qquad \text{Eq. 4}$$

where h_x is the maximum center-to-center horizontal spacing of cross tie or hoop legs.

Figure 15 presents the volumetric reinforcement ratio for confining reinforcement as a function of steel yield strength for the Column B cross section; Table 13 presents details of the transverse reinforcement designs for all column cross sections. For the column cross sections and concrete strengths considered, volumetric reinforcement ratios for Grade 60 reinforcement ranged from 1.9% (No. 5 ties at 4.5 in.) to 3.6% (No. 7 ties at 4.5 inches).

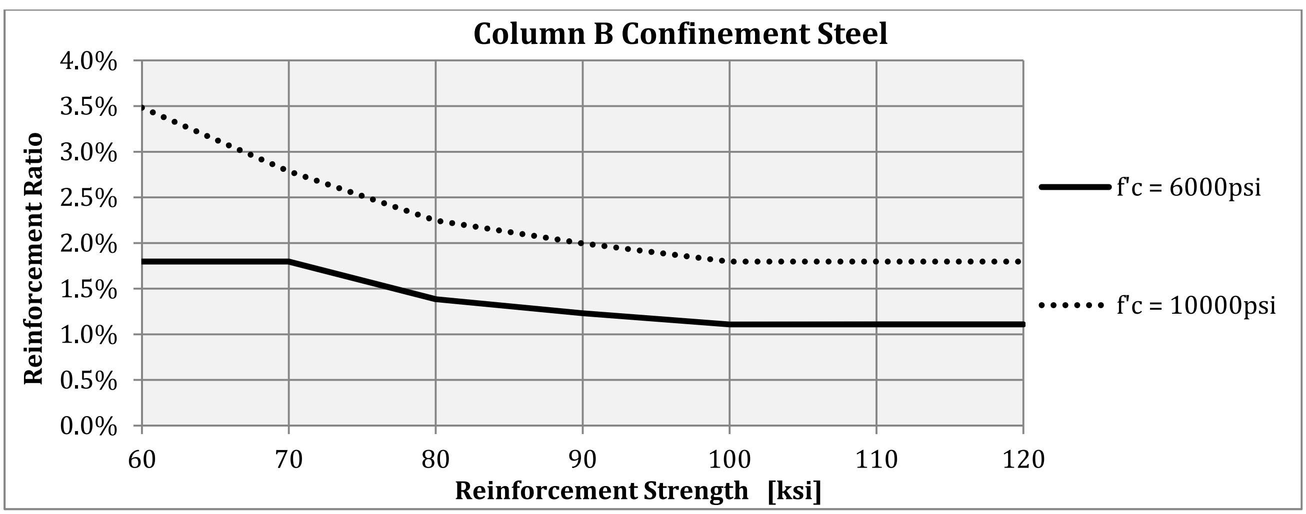


Figure 15 - Column Longitudinal Reinforcement Ratio Versus Steel Strength

Table 13 - Column Confinement Steel Results

Column A

	6000 psi			10000 psi		
	Tie Size	**s [in]**	**ρ_s**	**Tie Size**	**s [in]**	**ρ_s**
60	5	4.5	1.90%	6	4.0	3.29%
70	5	5.0	1.71%	6	4.5	2.92%
80	4	4.0	1.32%	6	5.0	2.63%
90	4	4.5	1.17%	5	4.0	2.14%
100	4	5.0	1.06%	5	4.5	1.90%
110	4	5.0	1.06%	5	5.0	1.71%
120	4	5.0	1.06%	5	5.0	1.71%

Column B

	6000 psi			10000 psi		
	Tie Size	**s [in]**	**ρ_s**	**Tie Size**	**s [in]**	**ρ_s**
60	5	5.0	1.80%	6	4.0	3.48%
70	5	5.0	1.80%	6	5.0	2.79%
80	4	4.0	1.39%	5	4.0	2.25%
90	4	4.5	1.23%	5	4.5	2.00%
100	4	5.0	1.11%	5	5.0	1.80%
110	4	5.0	1.11%	5	5.0	1.80%
120	4	5.0	1.11%	5	5.0	1.80%

Column C

	6000 psi			10000 psi		
	Tie Size	**s [in]**	**ρ_s**	**Tie Size**	**s [in]**	**ρ_s**
60	5	4.0	2.03%	7	4.5	3.64%
70	5	4.5	1.81%	6	4.0	3.12%
80	5	4.5	1.81%	6	4.5	2.77%
90	4	4.0	1.26%	6	4.5	2.77%
100	4	4.5	1.12%	5	4.0	2.03%
110	4	4.5	1.12%	5	4.5	1.81%
120	4	4.5	1.12%	5	4.5	1.81%

Observations

The data in Figure 15 and Table 13 show that in comparison with Grade 60 reinforcement the use of HSR results in a reduced volume of confining reinforcement. The volume of reinforcement required for confinement decreases with increasing steel yield strength up to the point at which the maximum spacing limit, defined by the geometric properties of the column, controls the design. For the column configurations considered here, the minimum spacing limit was typically reached for f_y = 100ksi.

SHEAR REINFORCEMENT

In regions of high seismicity, ACI Code Section 21.13 controls design of gravity column transverse reinforcement to meet shear strength requirements. Specific requirements apply depending on a variety of factors including (1) if shear reinforcement is designed for column ends where flexural yielding may occur under earthquake loading or for elsewhere along the length of the column where flexural yielding would not be expected and (2) how shear demand is determined. For the current study, shear reinforcement was designed for column ends to enable comparison with confining reinforcement. The Code allows that column shear demand used for design may be determined by the flexural capacity of the columns or the transverse members that frame into the column, whichever is less (Section 21.6.5.1). For gravity columns, this is typically the flexural capacity of the slabs. For the current study, shear demand induced by framing members was not known and was conservatively estimated by assuming that columns achieved the maximum factored flexural capacity, as taken from the P-M interaction curve, at the top and bottom of the column. Depending on how shear demand is determined, the Code allows that concrete may or may not be considered to contribute to column shear strength at the column ends where flexural yielding may occur under earthquake loading. For the current study, transverse reinforcement was designed to meet strength requirements for both cases: concrete contributing and not contributing to shear strength.

Figure 16 compares the transverse reinforcement ratio required at column ends as determined by confinement and shear strength requirements for all three column cross sections, both concrete compressive strengths, the range of steel yield strengths, and no concrete contribution to shear strength.

Observations

The data in Figure 16 show that the use of HSR offers the potential for reducing the volume of transverse reinforcement used in gravity column construction. The ratio of the required volume of HSR to Grade 60 reinforcement is approximately equal to the inverse of the ratio of the yield strengths of the reinforcing steel. If shear demand is computed such that concrete may be assumed to contribute to gravity column shear strength, then design of transverse reinforcement is controlled by the maximum spacing limit and the impact of steel strength on the required volume of transverse reinforcement is reduced.

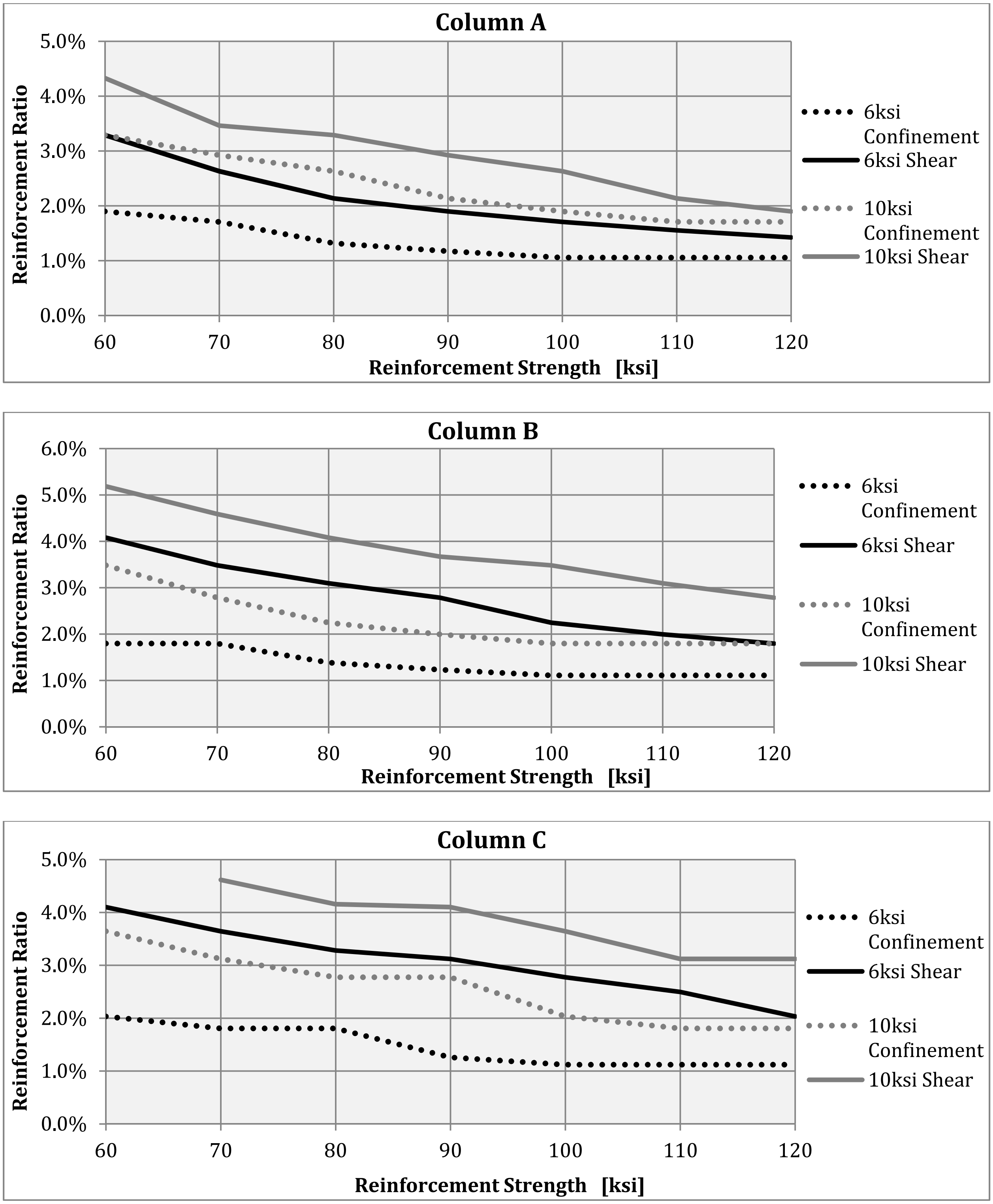


Figure 16 – Transverse Reinforcement Ratios versus Steel Strength for Different Column Geometries

CONCLUSIONS REGARDING DESIGN OF GRAVITY COLUMNS USING HSR

The use of HSR offers the potential for reducing the volume of longitudinal and confining reinforcement used in construction of gravity columns. However, the reduction in reinforcement achieved through use of HSR is limited by the Code required minimum longitudinal reinforcement ratio and by the Code specified maximum spacing limit for confining reinforcement. Specific observations and conclusions regarding the use of HSR in design of gravity columns are as follows:

1. The use of HSR can reduce the required volume of column longitudinal steel. Reduction in reinforcement volume is greatest for tension-controlled columns and limited for compression-controlled columns. In general, the volume of longitudinal reinforcement cannot be reduced below the Code required minimum of 1% (ACI Code Section 10.9).
2. The Code required minimum longitudinal reinforcement ratio of 1% is intended to ensure that load transfer to longitudinal reinforcement due to concrete creep under sustained loads does not result in yielding of the reinforcement. Given the higher yield strength of HSR, there is the potential for a reduced volume of HSR to result in performance and reliability that are comparable to that achieved using Grade 60 reinforcement and a 1% reinforcement ratio.
3. The use of HSR can reduce the required volume of confining reinforcement used in gravity columns. This reduction is limited by the maximum spacing determined by geometric properties of the column. For the most of the column designs considered here, a reduction in the required volume of confining reinforcement was realized when steel yield strength was increased from 60 ksi to 100 ksi. For yield strengths beyond 100 ksi, spacing limits controlled design of the confining steel and no additional reduction in steel volume was achieved when yield strength was increased.
4. Design of transverse reinforcement for gravity columns is rarely controlled by shear strength requirements. Design of transverse reinforcement may be determined by shear strength requirements if (i) maximum shear demand, as determined by the probable flexural strength of the column, is considered and (ii) the concrete contribute to column shear strength is ignored. Under these conditions, increasing steel strength can provide up to a 50% reduction in steel.

STRUCTURAL WALLS

The same process used to assess the impact of HSR on the design of gravity columns was used for structural walls. Two Grade 60 planar reference wall designs were considered. These designs employed a typical wall geometry, typical longitudinal reinforcement ratios for the boundary elements and web of the wall, and typical concrete compressive strengths. P-M interaction curves were generated for these reference walls. Longitudinal reinforcement was then redesigned, using HSR with a range of yield strengths, to achieve the same factored flexural strength as the Grade 60 designs at axial loads ranging from 10% to 30% of factored axial strength. P-M interaction curves were generated for the HSR designs. For Grade 60 and HSR designs, boundary element confining reinforcement was designed to meet Code requirements and horizontal shear reinforcement was designed for shear demands ranging from $V_u = 2A_{cv}\sqrt{f_c'}$ psi to $V_u = 7.5A_{cv}\sqrt{f_c'}$ psi.

LONGITUDINAL REINFORCEMENT

The same process used to assess the impact of HSR on the design of longitudinal steel in gravity columns was used for structural walls. P-M interaction curves were generated for reference walls with typical geometries and reinforcement ratios, typical concrete compressive strengths and Grade 60 reinforcement. Longitudinal reinforcement was then redesigned, using HSR with a range of yield strengths, to achieve the same factored flexural strength as the Grade 60 designs at axial loads ranging from 10% to 30% of factored axial strength. P-M interaction curves were generated for the HSR designs.

The Grade 60 reference walls were 20 ft. long and 2 ft. thick, with 3 ft. long boundary elements at each end of the wall containing 2.2% longitudinal reinforcement, and with a longitudinal reinforcement ratio for the web region of the wall of 0.5% (Figure 6). The reference walls employed concrete with compressive strengths of 6,000 psi and 10,000 psi. HSR wall designs employed steel with f_y= 80 ksi, 100 ksi and 120 ksi. For the HSR wall designs, boundary element reinforcement was designed to achieve the desired flexural strength; a web longitudinal reinforcement ratio of 0.5% was assumed for all designs.

For Grade 60 and HSR designs, P-M interaction curves were generated using the software spColumn (http://www.structurepoint.org). Appropriate strength reduction factors for tension-controlled (flexural members) and compression-controlled (compression members) response, φ = 0.9 and φ = 0.65 respectively, were considered in generating the curves. For each Grade 60 reference wall design, a P-M interaction curve was generated. The boundary element longitudinal reinforcement was then redesigned using HSR to achieve the same factored flexural strength at an axial load of 10% of the factored axial strength, $0.1\varphi A_g f_c'$, and a new P-M curve was generated for this cross-section design. The process of redesigning the section and generating new P-M curves was then repeated to achieve comparable flexural strength at axial loads equal to 20% and 30% of the factored axial strength of the Grade 60 design. Table 14 presents longitudinal reinforcement ratios for the different designs.

Table 14 - Wall Longitudinal Steel Results

	6000 psi						10000 psi					
	$A_{s\_10\%}$ [in²]	$\rho_{\_10\%}$	$A_{s\_20\%}$ [in²]	$\rho_{\_20\%}$	$A_{s\_30\%}$ [in²]	$\rho_{\_30\%}$	$A_{s\_10\%}$ [in²]	$\rho_{\_10\%}$	$A_{s\_20\%}$ [in²]	$\rho_{\_20\%}$	$A_{s\_30\%}$ [in²]	$\rho_{\_30\%}$
60	19.02	2.20%	19.02	2.20%	19.02	2.20%	19.02	2.20%	19.02	2.20%	19.02	2.20%
80	11.86	1.37%	11.86	1.37%	12.70	1.47%	11.86	1.37%	12.30	1.42%	12.30	1.42%
100	8.04	0.93%	8.04	0.93%	9.20	1.06%	8.04	0.93%	8.04	0.93%	8.56	0.99%
120	5.14	0.59%	5.00	0.69%	24.02	2.78%	5.14	0.59%	5.58	0.65%	28.08	3.25%

Observations

The impact of HSR on wall design is essentially the same as for gravity columns. For walls with low axial load demands exhibiting tension-controlled response (i.e. flexural members), the use of HSR reduces the volume of longitudinal reinforcement required to achieve the required flexural strength. The ratio of required HSR to Grade 60 longitudinal reinforcement is approximately equal to the inverse of the ratio of the yield strengths of the reinforcement. For walls with high axial load demands exhibiting compression-controlled response (i.e. compression members), response is primarily determined by geometry and concrete strength and designs utilizing HSR are insensitive to steel strength. For walls with axial loads that are in the transition from tension- to compression-controlled response, approximately 30% of the compression capacity for the wall configuration considered here, wall designed using HSR can require a significantly *larger* volume of longitudinal reinforcement to achieve the same *factored* flexural strength as a wall designed using Grade 60 reinforcement. As this is an artifact of the discrete transition from tension- to compression-controlled response, it is unlikely that this would control design using HSR.

While this study shows that the impact of HSR on design for the full range of axial load demands is comparable for walls and columns, the use of HSR likely has a much greater potential for reducing steel volumes used in construction of structural walls. This is due to fact that walls are typically proportioned to serve as part of the lateral load-resisting system, and often have relatively low axial load demands. This is contrary to gravity columns, which typically have much higher axial load demands. Thus, wall designs employed in practice could be expected to fall into the tension-controlled portion of the P-M interaction curve where the use of HSR can greatly reduce the required volume of longitudinal reinforcement.

BOUNDARY ELEMENT CONFINING REINFORCEMENT

For walls in regions of high seismicity, boundary element confining reinforcement must be provided to meet the requirements of ACI Code Section 21.6.4. This Section also controls design of confining reinforcement for gravity columns. Table 15 provides results for wall boundary element reinforcement designed to meet these requirements.

Table 15 - Wall Confinement Steel Results

Column A	6,000 psi			10,000 psi		
	Tie Size	s [in]	ρ_s	Tie Size	s [in]	ρ_s
60	5	6.0	2.19%	5	4.0	3.97%
80	4	5.0	1.60%	5	4.5	2.90%
100	4	6.0	1.32%	4	4.0	1.99%
120	4	6.0	1.23%	4	4.5	1.79%

Observations

The data in Table 15 show that in comparison with Grade 60 reinforcement, the use of HSR results in a reduced volume of boundary element confining reinforcement. The volume of reinforcement required decreases with increasing steel strength. Because it is possible to reconfigure boundary element confinement (i.e. remove cross ties), ACI Code requirements for spacing of confining steel do not limit the volume reduction that can be achieved using HSR.

SHEAR REINFORCEMENT

To investigate the impact of HSR on design of horizontal reinforcement to meet shear strength requirements, walls were designed for the full range of shear demands encountered in design: $V_u = 2A_{cv}\sqrt{f_c'}$ psi to $V_u = 7.5A_{cv}\sqrt{f_c'}$ psi. Horizontal reinforcement was assumed to be distributed in two curtains per Section 21.9.2.2. Shear capacity was defined per ACI Code Eq. 21-7, repeated here as Eq. 5:

$$V_n = A_{cv}\left(\alpha_c \lambda \sqrt{f_c'} + \rho_t f_y\right) \qquad \text{Eq. 5}$$

where A_{cv} is the area of wall resisting shear demand; α_c is the coefficient defining the contribution of concrete to wall strength, taken equal to 2.0 to represent the case of a slender wall with a height-to-length ratio greater than 2.0; λ is the lightweight modification factor, taken equal to 1.0 to represent normal weight concrete, and ρ_t is the ratio of transverse reinforcement to gross concrete area. Horizontal reinforcement was designed to achieve $\phi V_n \geq V_u$, where $\phi = 0.75$ per ACI Code Section 9.3.4 assuming that shear demand is determined on the basis of wall flexural strength. For low shear demands, design of horizontal reinforcement was controlled by the ACI Code specified minimum reinforcement ratio (Section 21.9.2.1). Shear design details are listed in Table 16

Observations

For moderate to high shear demands ($V_u = 4A_{cv}\sqrt{f'_c}$ psi to $V_u = 7.5A_{cv}\sqrt{f'_c}$ psi), increasing the steel strength reduces the amount of steel required to meet shear strength requirements. For low shear demand, $V_u \approx 2A_{cv}\sqrt{f'_c}$, however, the ACI Code minimum reinforcement ratio controls design and steel yield strength does not affect the design.

Table 16 - Wall Shear Steel Results

	Load	$V_u = 2A_{cv}\sqrt{f'_c}$			$V_u = 4A_{cv}\sqrt{f'_c}$			$V_u = 6A_{cv}\sqrt{f'_c}$			$V_u = 7.5A_{cv}\sqrt{f'_c}$		
		s [in]	Bar Size	ρ_t	s [in]	Bar Size	ρ_t	s [in]	Bar Size	ρ_t	s [in]	Bar Size	ρ_t
6,000 psi	60	6	4	0.25%[1]	8	6	0.46%	4	6	0.92%	3	6	1.22%
	80	6	4	0.25%[1]	11	6	0.33%	6	6	0.61%	4	6	0.92%
	100	6	4	0.25%[1]	14	6	0.26%	7	6	0.52%	5	6	0.73%
	120	6	4	0.25%[1]	14	6	0.25%[1]	9	6	0.41%	7	6	0.52%
10,000 psi	60	6	4	0.25%[1]	6	6	0.61%	3	6	1.22%	2	6	1.83%
	80	6	4	0.25%[1]	8	6	0.46%	4	6	0.92%	3	6	1.22%
	100	6	4	0.25%[1]	11	6	0.33%	6	6	0.61%	4	6	0.92%
	120	6	4	0.25%[1]	13	6	0.28%	7	6	0.52%	5	6	0.73%

Note: 1. longitudinal reinforcement design was controlled by ACI Code Section 21.9.2.1.

IMPACT OF HSR ON TOTAL STEEL WEIGHT FOR STRUCTURAL WALLS

Figure 17 shows the total weight of longitudinal, confining and shear reinforcement used in wall design as a function of steel strength; steel weight is normalized by the weight for design using Grade 60 reinforcement. These data show that increasing steel yield strength significantly reduces the required volume of reinforcing steel for steel yield strengths between 60 ksi and 100 ksi.

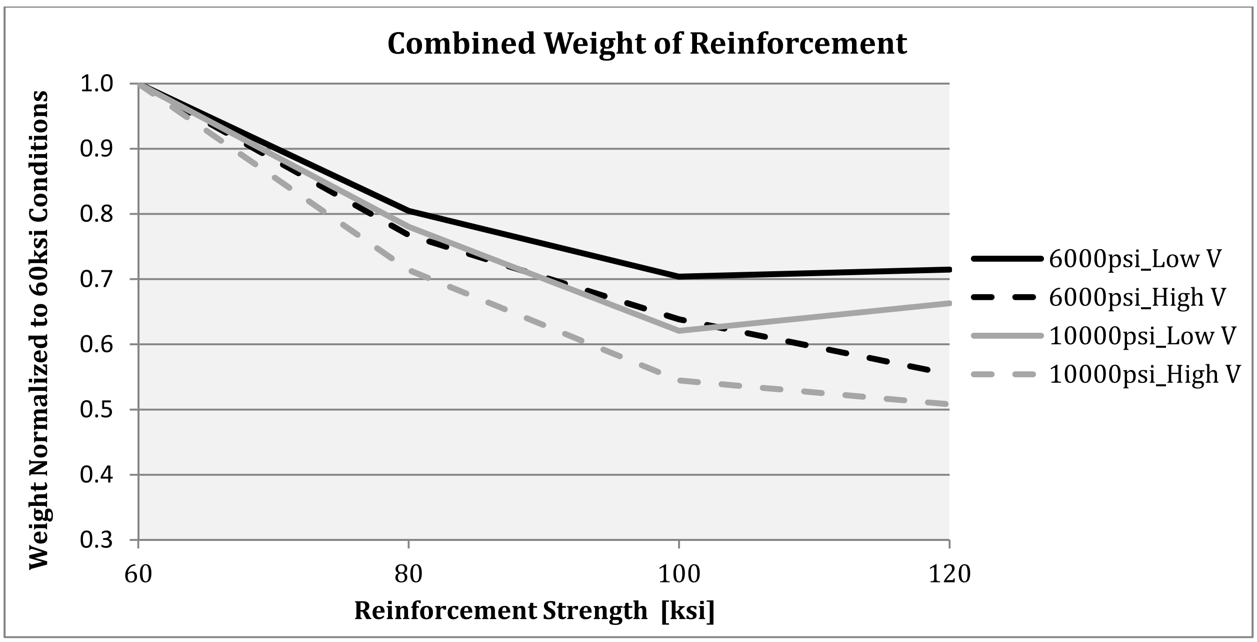


Figure 17 – Normalized Weight of Longitudinal, Confining, and Shear Reinforcement for Structural Walls with the Same Flexural Strength and Different Concrete Strengths and Shear Demands

CONCLUSIONS REGARDING DESIGN OF STRUCTURAL WALLS USING HSR

The use of HSR offers the potential for reducing the volume of reinforcement used in construction of structural walls. The greatest reduction is observed for wall with relatively low axial loads (less than 20% of factored axial strength) and moderate to high shear demands $\left(V_u \geq 4A_{cv}\sqrt{f_c'}\text{ psi}\right)$. Specific observations and conclusions regarding the use of HSR in design of gravity columns are as follows:

1. The use of HSR results in a reduced volume of boundary element longitudinal steel. Reduction in reinforcement volume is greatest for walls with low axial loads that meet ACI specifications for tension-controlled response and limited for walls with higher axial loads that meet ACI Code specifications for compression-controlled response.
2. The use of HSR results in a reduced volume of boundary element confining reinforcement. This reduction may be limited by the ACI Code maximum spacing limit. However, for the boundary element configurations considered in this study, it was possible to reconfigure boundary element configure reinforcement by removing cross ties and thereby realize reductions in steel volume for steel yields strengths up to 120 ksi.
3. For moderate to high shear demands $\left(V_u \geq 4A_{cv}\sqrt{f_c'} \text{ psi}\right)$, the use of HSR results in a reduced volume of horizontal reinforcement used in walls to meet shear strength requirements. For moderate to low shear demands $\left(V_u < 4A_{cv}\sqrt{f_c'} \text{ psi}\right)$, the ACI Code minimum reinforcement ratio controls the design, and the required volume of reinforcement is not affected by steel strength..
4. The use of HSR significantly reduces the total volume reinforcement used in walls. For steel yield strengths varying from 60 ksi to 100 ksi, the required volume of reinforcement decreases with increasing yield strength. For steel with f_y > 100 ksi, ACI Code minimum reinforcement requirements limit the impact of HSR.

CONCLUSIONS

FLEXURAL MEMBERS: SLABS AND BEAMS

The use of HSR offers the potential for reducing steel volumes for flexural members (slabs and beams). However, this is limited by ACI Code requirements on the minimum longitudinal reinforcement ratio and maximum spacing of longitudinal reinforcement. The use of HSR has minimal impact on member deflections as deflections are determined primary by the gross section dimension of the member. Thus, member thicknesses/depths remain essentially the same when HSR is used. With member gross dimension largely unaffected by steel yield strength, Code minimum longitudinal reinforcement ratios often control the design in regions of the member where strength demands are low or if f_y = 120 ksi reinforcement is used. To meet implied ACI Code limits on flexural crack widths and achieve the most efficient use of HSR, it may be necessary to use smaller, more closely spaced HSR bars.

COMBINED FLEXURAL AND AXIAL MEMBERS: GRAVITY COLUMNS

The use of HSR offers the potential for reducing steel volumes for gravity columns. The use of HSR has the greatest impact on design of confining reinforcement required at the ends of the column. In most cases, the ratio of the required volume of HSR to Grade 60 confining reinforcement is approximately equal to the inverse of the ratio of the yield strengths of the reinforcement. Shear demands for gravity columns are typically limited by slab flexural capacity such that column shear demands are low and the use of HSR does not significantly reduced the volume of shear reinforcement. The design of gravity column longitudinal reinforcement is typically controlled by the ACI Code required minimum reinforcement ratio, in which case the use of HSR does not reduce the longitudinal steel volume. If this requirement is ignored, the use of HSR significantly reduces longitudinal reinforcement volume for columns with low axial loads that exhibit tension-controlled response.

COMBINED FLEXURAL AND AXIAL MEMBERS: STRUCTURAL WALLS

The use of HSR offers the potential for significantly reducing steel volumes for structural walls. This is particularly true for walls with low axial loads that meet ACI Code requirements for tension-controlled response and moderate to high shear demand $\left(V_u \geq 4A_{cv}\sqrt{f_c'} \text{ psi}\right)$. For these walls and the design of longitudinal, confining and shear reinforcement, the ratio of the required volume of HSR to Grade 60 reinforcement is approximately equal to the inverse of the ratio of the yield strengths of the reinforcement. For walls with high axial loads that do not meet ACI Code requirements for tension-controlled response, the use of HSR may not affect or may even increase the required longitudinal reinforcement ratio in comparison with Grade 60 reinforcement. For walls with moderate to low shear demand, the ACI Code minimum horizontal reinforcement ratio may control design of shear reinforcement; thus, the use of HSR has no impact on the required volume of shear reinforcement.

RECOMMENDED MAXIMUM STEEL STRENGTHS

The results of this study show that if ACI Code limits on steel yield strength are ignored, HSR can be used to reduce steel volumes for RC building components. However, results show also that for some components, there is a limit beyond which increasing the yield strength of the reinforcing steel does not result in reduced steel volume due to serviceability requirements. Table 17 lists the recommended maximum reinforcement yield strength for use in design of the components considered in this study; the use of reinforcing steel with yield strengths in excess of this value would not be expected to result in further reduction in the volume of reinforcement. Recommendations are limited to the range of steel strengths considered in this study $\left(60\ ksi\ \leq\ f_y \leq 120\ ksi\right)$.

Table 17 – Recommended Maximum Steel Yield Strength for Use in Design

Component	Recommended Maximum F_y		Note
One-Way Slabs	Longitudinal –	80 ksi	Use of 120 ksi steel requires the use of a larger number of smaller bars (No. 3) to meet maximum flexural crack width limits.
Two-Way Slabs	Longitudinal –	60 ksi	Use of HSR requires the use of a larger number of smaller bars (No. 3) to meet maximum flexural crack width limits.
Beams	Longitudinal – Shear –	120 ksi 60 ksi	
Gravity Columns, low axial & high shear and moment demand	Longitudinal – Confinement – Shear –	120 ksi 100 ksi 120 ksi	Recommendations are appropriate for columns with axial load less than $0.2f_c'A_g$ subjected to high shear demand.
Gravity Columns, high axial & low shear and moment demand	Longitudinal – Confinement – Shear –	60 ksi 100 ksi 60 ksi	Recommendations are appropriate for columns with axial load greater than $0.3f_c'A_g$ subjected to low shear demand.
Structural Walls	Longitudinal – Confinement – Shear –	120 ksi 120 ksi 120 ksi	Recommendations are appropriate for walls with axial load less than $0.2f_c'A_g$ subjected to shear demand in excess of $4A_{cv}\sqrt{f_c'}$ psi.

RECOMMENDATIONS FOR FUTURE RESEARCH

The study presented here addresses design of gravity load-resisting components and structural walls using HSR and current ACI Code requirements. The results of this study suggest the following future research activities to support ACI Code changes to realize the full potential of HSR.

1. The sizing of slabs and beams is controlled by deflection requirements under service-level loading. The results of this study suggest that deflections are primarily determined by gross-section properties such that the use of HSR has minimal impact on member sizing. These results

are based on the assumption that cracking due to shrinkage, temperature, and creep effects does not significantly reduce flexural stiffness over a large region of the system. Additional research is needed to verify this assumption and verify the results suggested by this study.

2. Design of flexural members using HSR may require the use of smaller, more closely spaced longitudinal reinforcing bars to meet ACI Code requirements intended to limit maximum flexural crack widths. These requirements are "intended to limit surface cracks to a width that is generally acceptable in practice" (ACI 318 2011), and "research shows that corrosion is not clearly correlated with surface crack widths in the range normally found with reinforcement stresses at service load levels." (ACI 318 2011). Recent research also indicates that current methods (Frosh 1999) are appropriate for quantifying flexural crack widths for members reinforced with HSR (Harries et al. 2012). Given the above and the potential for flexural crack width requirements to impact design using HSR, additional research addressing the Code intended limit on flexural crack widths is warranted. This research would likely take the form of a survey of building owners and current construction to determine (i) what constitutes acceptable flexural cracking and (ii) if current Code requirements and construction techniques result in acceptable flexural cracking.
3. Design of flexural members using HSR may be controlled by ACI Code minimum reinforcement ratios intended to ensure that the nominal flexural strength of the component exceeds the cracking strength. These Code provisions were developed considering lower strength reinforcement and should be evaluated for HSR.
4. Design of gravity column longitudinal reinforcement is typically controlled by the ACI Code specified minimum reinforcement ratio for compression members, $\rho_l = 1\%$. This minimum is intended to ensure that longitudinal reinforcement does not yield under sustained axial load as steel stress increases due to concrete creep. This Code requirement is based on laboratory tests conducted in the 1930s, and the initial requirement included in the Code specified a minimum reinforcement ratio of $\rho_l = 0.5\%$ for tied columns. Given the higher yield strength of HSR, there is the potential for columns constructed of HSR with $\rho_l < 1\%$ to achieve the same level of performance and safety as those constructed of lower strength reinforcement with $\rho_l = 1\%$. Laboratory testing of columns under sustained axial load is required to investigate this issue and develop recommendations for minimum reinforcement for compression members that are a function of longitudinal reinforcement yield strength. Laboratory tests should investigate the following design parameters: column geometry, longitudinal reinforcement ratio, steel yield strength, and axial load ratio.
5. For slabs and walls, design using HSR may be controlled by ACI Code minimum reinforcement ratios and maximum reinforcement spacing requirements that are intended to limit the width of cracks resulting from shrinkage and temperature effects (slabs and walls), shear (walls) and local demands (slabs). These requirements are intended to ensure that reinforcement does not yield at the crack, and thereby allow for wide cracks to develop. Minimum reinforcement ratios were developed and evaluated considering lower grade reinforcing steels. Given the higher yield strength of HSR and the objective of preventing steel yielding at the crack, additional research to assess these requirements for HSR is warranted.

The above recommendations for future research follow directly from the current study. It is expected that Code changes to enable use of HSR will require additional research also to demonstrate that components constructed using HSR can provide overall performance comparable to that achieved using lower strength steel. These activities would necessarily address behavior under sustained gravity and earthquake loads.

REFERENCES

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