United States Department of Agriculture

Soil Conservation Service

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April 12, 1983

DESIGN NOTE NO. 21

SUBJECT: ENG - SUBSTITUTION OF HIGHER STRENGTH STEELS IN REINFORCED CONCRETE

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Purpose. To distribute Design Note No. 21, "Considerations on the Substitution of Higher Strength Steels in Reinforced Concrete Construction."

Effective Date. Effective when received.

Explanation. Questions have risen concerning the permissibility of substituting, during construction, higher strength reinforcing steel than was assumed in design. Some have not recognized that there may be a potential problem with respect to substituting higher strength steel.

The design note discusses the question and provides guidance in reaching a decision. Example problems illustrate that the decision to substitute should not be based on the presumption that a higher yield strength steel must produce a better design. Section strength and ductility and structure load carrying capacity should also be considered.

Interestingly, the substitution question is particularly germane because of our ongoing transition from working stress design to strength design methods. The question was of little concern when all designing was performed by working stress design in accordance with SCS allowable values.

Filing Instructions. File with other criteria pertaining to the design and/or construction of reinforced concrete structures.

Distribution. The design note should be of interest to both design engineers and construction engineers. Initial distribution (shown on reverse side) to each State and NTC is sufficient to provide a copy for each employee who is a practicing professional engineer. Additional copies may be obtained from Central Supply by ordering item No. DN-21.

Thomas G. Shifflet, Actg

PAUL M. HOWARD
Deputy Chief for Technology
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Enclosure

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SOIL CONSERVATION SERVICE

DESIGN NOTE NO. 21

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REMARKS

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April 1, 1983

Design Note No. 21*

Subject: Considerations on the Substitution of Higher Strength Steels in Reinforced Concrete Construction

INTRODUCTION

Questions have risen concerning the permissibility of substituting, during construction, higher strength reinforcing steel than was assumed in design.

Usually the question arises becauses Grade 40 steel, i.e, f_y = 40 ksi, was assumed in design — either because of designer preference or criteria requirement — but Grade 40 steel was found not readily available at the time of construction. Permission is then sought to substitute either Grade 50 or more likely Grade 60 steel since it, in particular, is generally more readily available from suppliers.

Some might want to adopt the seemingly obvious premise that a higher strength steel must yield a better design than can the lower strength steel. This is too simplistic. The decision to substitute should not be based solely on the ratio of the yield strengths of the steels. The problem is treated in some detail in the work that follows.

As preliminaries to detailed discussion, the points below should be recognized.

1. NEM §536.20(c)(3) and §536.20(d)(2)(*i*) indicate the grade of steel should be specified for the construction. The Instructions for use of Guide Construction Specification 34. Steel Reinforcement state that the type and grade of steel should be included in the drawings or specifications if it is necessary to restrict the contractor's choice from the values in Guide Material Specification 539. Steel Reinforcement.
2. The original design, for the lower strength steel, must be adequate in all respects, satisfying all requirements for the original lower strength steel. The basic question is — can a higher strength steel be properly substituted for a lower strength steel? That is, the substitution of a higher strength steel should not be an attempt to upgrade an unsatisfactory design.
3. Deformed bars with yield strengths in excess of 60 ksi are usually not permitted and hence are not considered herein.

*Prepared by Edwin S. Alling, Head, Design Unit, Engineering Division, Lanham, Maryland

QUESTIONS AND SITUATIONS

The overall subject of substituting higher strength steel can be framed in the form of three questions. These are:

1. Is the substitution of Grade 50 or Grade 60 bars acceptable when the design assumed Grade 40?

2. What is the effect of substituting Grade 50 or Grade 60 bars when the design assumed Grade 40?

3. What should be done, if anything, when Grade 50 or Grade 60 bars are substituted for Grade 40?

Three situations, in terms of structure environment class, are encountered. These are:

1. The design is for a Service hydraulic structure. Hence, in accordance with NEM §536.20(c)(2) the design yield strength is taken as $f_y = 40$ ksi for Grades 40, 50, or 60 steels. Thus $f_y = 40$ ksi and the maximum design steel ratio permitted is $\rho_{max} = \rho_{shy_{40}}$.

    a. Moment redistribution was not employed in the design. What are the consequences of using Grade 50 or 60?

    b. Moment redistribution was employed in the design. Hence Grade 40 steel should be specified for construction. What are the consequences of using Grade 50 or 60?

2. The design is for other structures — with uncontrolled environment. Hence the maximum design steel ratio permitted is $\rho_{max} = 0.50\overline{\rho}_b$. The design yield strength may be taken as $f_y = 40$, 50, or 60 ksi in accordance with the grade of steel specified.

    a. Moment redistribution was not employed in the design. If Grade 40 was specified, what are the consequences of using Grade 50 or 60?

    b. Moment redistribution was employed in the design. If Grade 40 was specified, what are the consequences of using Grade 50 or 60?

3. The design is for other structures — with controlled environment. Hence the maximum design steel ratio permitted is $\rho_{max} = 0.75\overline{\rho}_b$. The design yield strength may be taken as $f_y = 40$, 50, or 60 ksi in accordance with the grade specified. If Grade 40 was specified, what are the consequences of using Grade 50 or 60?

INVESTIGATIONS, PROBLEMS, AND OBSERVATIONS

In investigating the consequences of using a higher strength steel for construction than was assumed in design, the effects on a number of items are considered. These include:

1. flexural strength of critical sections,
2. rotation capacity of critical sections,
3. load carrying capacity of the system, and
4. failure mode.

Where flexural shear and/or torsional shear are significant, these items also need consideration. A number of example problems are worked through below. They illustrate typical thought processes. The conclusions drawn may not be general, design specific investigations will often be warranted.

All examples use a common base. Thus:

Grade 40 steel assumed in design.

The structure is a fixed ended, uniformly loaded, prismatic beam.

f'_c = 4000 psi

b = 12 inches

d = 10 inches

The examples treat full moment redistribution* for purposes of illustration and emphasis. Note that currently ACI 318-77, section 8.4 restricts application of moment redistribution to a maximum change of negative support moments, increase or decrease, of 20% based on the steel ratio.

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*This design note should not be construed as advocating the use, or non-use, of the moment redistribution concept. The design note attempts to illustrate possible consequences of the substitution of higher strength steel if moment redistribution has not been used, or has been used.

By SCS criteria, redistribution of moments is permissible under certain conditions. The design note does show that moment redistribution should not be used in some situations. The ACI has recognized, for a number of years, the redistribution of moments that can occur in indeterminate r/c structures, see:

ACI 318-63, sections 1502(d), 2103(c), 2104(f)5
ACI 318-71, sections 8.6, 13.3.4.6
ACI 318-77, sections 8.4, 13.6.7

Also see corresponding code commentaries, for example, 1977 commentary on section 8.4.

Use of moment redistribution will not result in reinforcement savings if the design requires consideration of only one load condition. Here the steel will only be distributed differently between critical negative and positive plastic hinge locations. Use of moment redistribution can only result in less total steel when the design requires consideration of more than one loading condition.

Table 1

Strength Design
Flexure Steel Ratios.

Grade of Steel	Class of Concrete	$\bar{\rho}_b$	$0.75\bar{\rho}_b$	$0.5\bar{\rho}_b$	$0.25\bar{\rho}_b$	ρ_{shy}	$\rho_{shy}/\bar{\rho}_b$
60	6000	0.03773	.02830	0.01886	.00943	0.01368	0.36
	5000	0.03354	.02516	0.01677	.00839	0.01073	0.32
	4000	0.02851	.02138	0.01425	.00713	0.00795	0.28
	3000	0.02138	.01604	0.01069	.00535	0.00538	0.25
	2500	0.01782	.01337	0.00891	.00446	0.00419	0.24
50	6000	0.04858	.03644	0.02429	.01215	0.01844	0.38
	5000	0.04318	.03239	0.02159	.01080	0.01451	0.34
	4000	0.03671	.02753	0.01835	.00918	0.01080	0.29
	3000	0.02753	.02065	0.01376	.00688	0.00735	0.27
	2500	0.02294	.01721	0.01147	.00574	0.00574	0.25
40	6000	0.06551	.04913	0.03275	.01638	0.02629	0.40
	5000	0.05823	.04367	0.02911	.01456	0.02079	0.36
	4000	0.04949	.03712	0.02475	.01238	0.01556	0.31
	3000	0.03712	.02784	0.01856	.00928	0.01066	0.29
	2500	0.03093	.02320	0.01547	.00774	0.00837	0.27

Table 2

Working Stress Design
Flexure Steel Ratios, ρ_{bwsd}

Grade of Steel	Class of Concrete	$f_c = 0.45 f'_c$, $f_s = 24000$	$f_c = 0.45 f'_c$, $f_s = 20000$	$f_c = 0.40 f'_c$, $f_s = 24000$	$f_c = 0.40 f'_c$, $f_s = 20000$
60	6000	0.02375	-	0.01969	0.02629
	5000	0.01876	-	0.01551	0.02078
	4000	0.01402	-	0.01156	0.01556
	3000	0.00958	-	0.00787	0.01066
	2500	0.00752	-	0.00616	0.00837
50	6000	-	0.03154	-	0.02629
	5000	-	0.02501	-	0.02078
	4000	-	0.01878	-	0.01556
	3000	-	0.01292	-	0.01066
	2500	-	0.01017	-	0.00837
40	6000	-	0.03154	-	0.02629
	5000	-	0.02501	-	0.02078
	4000	-	0.01878	-	0.01556
	3000	-	0.01292	-	0.01066
	2500	-	0.01017	-	0.00837

Example 1A

Design is for a Service hydraulic structure.

Using $\rho = \rho_{shy_{40}}$, find: A_s, M_n and ε_s.

Thus

$$\rho = \rho_{shy} = 0.40 * \frac{4}{40}\left(\frac{1}{1 + \frac{1.25 * 40}{7.96 * 4}}\right) = 0.01556$$

$$A_s = \rho bd = 0.01556 * 12 * 10 = 1.867 \text{ sq. in.}$$

$$a = \frac{f_y A_s}{0.85 f'_c b} = \frac{40 * 1.867}{0.85 * 4 * 12} = \frac{74.68}{40.8} = 1.830 \text{ in.}$$

$$M_{n_{40}} = f_y A_s (d - a/2) = \frac{40 * 1.867}{12}(10 - 1.830/2) = 56.54 \text{ ft kips}$$

$$c = a/0.85 = 1.830/0.85 = 2.153 \text{ in.}$$

Steel unit strain at failure is a measure of the rotation capacity of the section. The failure strain is:

$$\varepsilon_s = 0.003\left(\frac{10 - 2.153}{2.153}\right) = 0.01093$$

The ratio of failure strain to yield strain is a measure of the ductility of the section, here

$$\varepsilon_y = \frac{40000}{29000000} = 0.00138$$

$$\varepsilon_s/\varepsilon_y = \frac{0.01093}{0.00138} = 7.92$$

If moment redistribution is not employed for this design, the uniform load capacity of this beam is:

$$w_{u_{40}} = \frac{12\ M_{n_{40}}}{\ell^2}$$

If full moment redistribution was permitted and employed for this design, the uniform load capacity of this beam is:

$$w_{u_{40}} = \frac{16\ M_{n_{40}}}{\ell^2}$$

As stated previously, ACI 318-77 limits moment redistribution to a maximum increase or decrease in elastic support moments of 20 percent. The uniform load capacity, under this maximum limitation, is:

$$w_{u_{40}} = \frac{1.20 * 12\ M_{n_{40}}}{\ell^2} = \frac{14.4\ M_{n_{40}}}{\ell^2}$$

Example 1B

Same as Example 1A except A_s given as 1.867 sq. in. and Grade 50 steel was supplied.

Find: ρ, ρ_{shy}, $\bar{\rho}_b$, $0.5\bar{\rho}_b$, M_n, and ε_s.

Thus

$$\rho = \frac{1.867}{120} = 0.01556$$

$$\rho_{shy} = 0.4 * \frac{4}{50}\left(\frac{1}{1 + \frac{1.25 \times 50}{7.96 * 4}}\right) = 0.01080$$

$$\bar{\rho}_b = 0.85 * 0.85 * \frac{4}{50}\left(\frac{87}{87 + 50}\right) = 0.03671$$

$$0.50\bar{\rho}_b = 0.01835$$

$\rho_{shy} < \rho < 0.50\bar{\rho}_b$ $\therefore$ this section would not be acceptable as a Service hydraulic structure if design were based on $f_y = 50$ ksi.

$$a = \frac{50 * 1.867}{0.85\ f'_c b} = \frac{93.35}{40.8} = 2.334 \text{ in.}$$

$$M_{n_{50}} = \frac{50 \times 1.867}{12}(10 - 2.334/2) = 68.71 \text{ ft kips}$$

$$c = 2.334/0.85 = 2.746 \text{ in.}$$

$\varepsilon_s = 0.003\left(\frac{10 - 2.746}{2.746}\right) = 0.00792$, thus rotation capacity of the section is less than in example 1A.

$$\varepsilon_y = 50/29000 = 0.00172$$

$\varepsilon_s/\varepsilon_y = 0.00792/0.00172 = 4.60$, thus the ductility of the section is less than in example 1A.

The ratio of the yield strengths is:

$$\frac{f_{y_{50}}}{f_{y_{40}}} = \frac{50}{40} = 1.25$$

The ratio of the flexural strengths for the sections of examples 1A and 1B is:

$$\frac{M_{n_{50}}}{M_{n_{40}}} = \frac{68.71}{56.54} = 1.22$$

$\rho < 0.50\bar{\rho}_b$ $\therefore$ moment redistribution would be possible by ACI for this section for $f_y = 50$ ksi

Since moment redistribution is possible, the ratio of the load carrying capacities for the beams of examples 1A and 1B is the same whether or not moment redistribution was employed for the two beams. The ratio is the same as the flexural strength ratio.
That is:

$$\frac{w_{u_{50}}}{w_{u_{40}}} = \frac{\dfrac{12\ M_{n_{50}}}{\ell^2}}{\dfrac{12\ M_{n_{40}}}{\ell^2}} = \frac{\dfrac{16\ M_{n_{50}}}{\ell^2}}{\dfrac{16\ M_{n_{40}}}{\ell^2}} = \frac{M_{n_{50}}}{M_{n_{40}}} = 1.22$$

if full moment redistribution employed

if moment redistribution not employed

Example 1C

Same as example 1A except A_s given as 1.867 sq. in. and Grade 60 steel was supplied.

Find: ρ, ρ_{shy}, $\bar{\rho}_b$, $0.50\bar{\rho}_b$, $0.75\bar{\rho}_b$, M_n and ε_s.

Thus

$$\rho = \frac{1.867}{120} = 0.01556$$

$$\rho_{shy} = 0.4 * \frac{4}{60}\left(\frac{1}{1 + \frac{1.25 * 60}{7.96 * 4}}\right) = 0.00795$$

$$\bar{\rho}_b = 0.85 * 0.85 * \frac{4}{60}\left(\frac{87}{87 + 60}\right) = 0.02851$$

$$0.50\bar{\rho}_b = 0.01425$$

$$0.75\bar{\rho}_b = 0.02138$$

$0.50\bar{\rho}_b < \rho < 0.75\bar{\rho}_b$ $\therefore$ this section would not be acceptable as a <u>Service hydraulic structure</u> if design were based on f_y = 60 ksi. Further, moment redistribution is not permitted by ACI for f_y = 60 ksi.

$$a = \frac{60 \times 1.867}{40.8} = 2.746$$

$$M_{n_{60}} = \frac{60 \times 1.867}{12}(10 - 2.746/2) = 80.53 \text{ ft kips}$$

$$c = 2.746/0.85 = 3.231 \text{ in}$$

$$\varepsilon_s = 0.003\left(\frac{10 - 3.231}{3.231}\right) = 0.00629$$

$$\varepsilon_y = 60/29000 = 0.00207$$

$$\varepsilon_s / \varepsilon_y = 0.00629/0.00207 = 3.04$$

The ratio of the yield strengths is

$$\frac{f_{y_{60}}}{f_{y_{40}}} = \frac{60}{40} = 1.50$$

If moment redistribution was not employed in design, the ratio of the flexural strengths and the ratio of the load carrying capacities for the beams of example 1A and 1C are the same. That is:

$$\frac{M_{n_{60}}}{M_{n_{40}}} = \frac{80.53}{56.54} = 1.42$$

and

$$\frac{w_{u_{60}}}{w_{u_{40}}} = \frac{\dfrac{12\ M_{n_{60}}}{\ell^2}}{\dfrac{12\ M_{n_{40}}}{\ell^2}} = \frac{M_{n_{60}}}{M_{n_{40}}} = 1.42$$

If full moment redistribution was employed in design assuming $f_y = 40$, but moment redistribution is not possible with $f_y = 60$ ksi, then the ratio of the load carrying capacities for the beams of example 1A and 1C, assuming no redistribution, is:

$$\frac{w_{u_{60}}}{w_{u_{40}}} = \frac{\dfrac{12\ M_{n_{60}}}{\ell^2}}{\dfrac{16\ M_{n_{40}}}{\ell^2}} = \frac{12}{16} * \frac{M_{n_{60}}}{M_{n_{40}}} = \frac{12}{16} * 1.42 = 1.07$$

or

$w_{u_{60}} \simeq w_{u_{40}}$ $\therefore$ under these assumptions, substitution of the higher strength steel would not significantly increase the load carrying capacity of the system.

Examples 1A, 1B, and 1C deal with a Service hydraulic structure with a design steel ratio $\rho = \rho_{shy_{40}}$. The examples illustrate that while substitution of higher strength steels produce steel ratios that exceed ρ_{shy} values for the higher strength steels, the steel ratios are less than $0.75\overline{\rho}_b$ for the higher strength steels. Therefore the failure mode remains well within the desired ductile range. The examples indicate that, at least for some cases, moment redistribution should not be employed if higher strength steels are permitted to be supplied for construction. Notice that although moment redistribution would seem permissible for $f_y = 50$ ksi steel in example 1B, moment redistribution would not be acceptable for all f_c' values in combination with $f_y = 50$ ksi. These examples verify that the criteria of NEM §536.20(c)(2) in concert with the criteria of NEM §536.20(d)(2)(*i*) yield acceptable designs.

Example 2A

Design is for other structures — with uncontrolled environment.

Using $\rho = 0.50\bar{\rho}_{b_{40}}$, find $\bar{\rho}_b$, A_s, M_n, and ε_s.

Thus

$$\bar{\rho}_b = 0.85 * 0.85 * \frac{4}{40}\left(\frac{87}{87 + 40}\right) = 0.04949$$

$$0.50\bar{\rho}_b = 0.02475$$

$$A_s = 0.02475 * 12 * 10 = 2.970 \text{ sq. in.}$$

$$a = \frac{40 * 2.970}{40.8} = 2.911 \text{ in.}$$

$$M_{n_{40}} = \frac{40 \times 2.970}{12}(10 - 2.911/2) = 84.59 \text{ ft kips}$$

$$c = 2.911/0.85 = 3.425 \text{ in.}$$

$$\varepsilon_s = 0.003\left(\frac{10 - 3.425}{3.425}\right) = 0.00576, \text{ compare with example 1A.}$$

$$\varepsilon_y = 40/29000 = 0.00138$$

$$\varepsilon_s/\varepsilon_y = 0.00576/0.00138 = 4.17, \text{ compare with example 1A.}$$

Without moment redistribution:

$$w_{u_{40}} = \frac{12\ M_{n_{40}}}{\ell^2}$$

With full moment redistribution:

$$w_{u_{40}} = \frac{16\ M_{n_{40}}}{\ell^2}$$

Example 2B

Same as example 2A except A_s given as 2.970 sq. in. and Grade 50 steel was supplied.

Find: ρ, $\bar{\rho}_b$, $0.50\bar{\rho}_b$, $0.75\bar{\rho}_b$, M_n, and ε_s

Thus:

$$\rho = \frac{2.970}{120} = 0.02475$$

$$\bar{\rho}_b = 0.03671$$

$$0.50\bar{\rho}_b = 0.01836$$

$$0.75\bar{\rho}_b = 0.02753$$

$0.05\bar{\rho}_b < \rho < 0.75\bar{\rho}_b$ $\therefore$ this section would not be acceptable for <u>other structures — with uncontrolled environment</u> if design were based on $f_y = 50$ ksi. Also, moment redistribution is not permitted by ACI for $f_y = 50$ ksi.

$$a = \frac{50 * 2.970}{40.8} = 3.640 \text{ in}$$

$$M_{n_{50}} = \frac{50 * 2.970}{12}(10 - 3.640/2) = 101.23 \text{ ft kips}$$

$$c = 3.640/0.85 = 4.282 \text{ in}$$

$$\varepsilon_s = 0.003\left(\frac{10 - 4.282}{4.282}\right) = 0.00401$$

$$\varepsilon_y = 50/29000 = 0.00172$$

$$\varepsilon_s/\varepsilon_y = 0.00401/0.00172 = 2.33$$

If moment redistribution was not employed in design, the ratio of the flexural strengths and the ratio of the load carrying capacities for the beams of example 2A and 2B are the same. That is:

$$\frac{M_{n_{50}}}{M_{n_{40}}} = \frac{101.23}{84.59} = 1.20$$

and

$$\frac{w_{u_{50}}}{w_{u_{40}}} = \frac{\dfrac{12\ M_{n_{50}}}{\ell^2}}{\dfrac{12\ M_{n_{40}}}{\ell^2}} = \frac{M_{n_{50}}}{M_{n_{40}}} = 1.20$$

If full moment redistribution was employed in design assuming $f_y = 40$, but moment redistribution is not possible with $f_y = 50$ ksi, then the ratio of the load carrying capacities for the beams of example 2A and 2B, assuming no redistribution, is:

$$\frac{w_{u_{50}}}{w_{u_{40}}} = \frac{\dfrac{12\ M_{n_{50}}}{\ell^2}}{\dfrac{16\ M_{n_{40}}}{\ell^2}} = \frac{12}{16} * \frac{M_{n_{50}}}{M_{n_{40}}} = \frac{12}{16} * 1.20 = 0.900$$

or

$w_{u_{50}} < w_{u_{40}}$ $\therefore$ under these assumptions, substitution of the higher strength steel would result in a decrease in load carrying capacity of the system.

Example 2C

Same as example 2A except A_s given as 2.970 sq. in. and Grade 60 steel was supplied.

Find: ρ $\bar{\rho}_b$, $0.75\bar{\rho}_b$, M_n, and ε_s

Thus:

$\rho = 2.970/120 = 0.02475$

$\bar{\rho}_b = 0.02851$

$0.75\bar{\rho}_b = 0.02138$

$0.75\bar{\rho}_b < \rho < \bar{\rho}_b$ $\therefore$ design would not be acceptable for any class of environment if design were based on f_y = 60 ksi. Further, this section has insufficient ductility by ACI.

$$a = \frac{60 \times 2.970}{40.8} = 4.368 \text{ in.}$$

$$M_{n_{60}} = \frac{60 * 2.970}{12}(10 - 4.368/2) = 116.07 \text{ ft kips}$$

$$c = 4.3688/0.85 = 5.139 \text{ in}$$

$$\varepsilon_s = 0.003(\frac{10 - 5.139}{5.139}) = 0.00284$$

$$\varepsilon_y = 60/29000 = 0.00207$$

$$\varepsilon_s/\varepsilon_y = 0.00284/0.00207 = 1.37$$

If moment redistribution was not employed in design

$$\frac{w_{u_{60}}}{w_{u_{40}}} = \frac{M_{n_{60}}}{M_{n_{40}}} = \frac{116.07}{84.59} = 1.37$$

If full moment redistribution was employed in design assuming f_y = 40, but moment redistribution is not possible with f_y = 60 ksi, then the ratio of the load carrying capacities for the beams of examples 2A and 2C, assuming no redistribution is:

$$\frac{w_{u_{60}}}{w_{u_{40}}} = \frac{\dfrac{12\ M_{n_{60}}}{\ell^2}}{\dfrac{16\ M_{n_{40}}}{\ell^2}} = \frac{12}{16} * \frac{M_{n_{60}}}{M_{n_{40}}} = \frac{12}{16} * 1.37 = 1.03$$

or

$w_{u_{60}} \simeq w_{u_{40}}$ $\therefore$ under these assumptions, substitution of the higher strength steel would result in essentially no increase in load carrying capacity of the system.

Examples 2A, 2B, and 2C deal with other structures — with uncontrolled environment with a design steel ratio $\rho = 0.50\overline{\rho}_{b_{40}}$. The examples illustrate that substitution of higher strength steels produces steel ratios that exceed the ACI limit for moment redistribution and often steel ratios that result in a lack of sufficient ductility. If moment redistribution is employed in the design assuming a low strength steel, but a higher strength steel is supplied in construction, then the ratio of the load carrying capacities of the system will be less than the ratio of the flexural strengths. In extreme cases, the load carrying capability of the system may actually decrease.

Example 3A

Design is for other structure — with controlled environment.

Using $\rho = 0.75\ \overline{\rho}_{b_{40}}$, find $\overline{\rho}_b$, A_s, M_n, and ε_s.

Thus:

$$\overline{\rho}_b = 0.04949$$

$$0.75\ \overline{\rho}_b = 0.03712$$

$$A_s = 0.03712 * 120 = 4.454 \text{ sq. in.}$$

$$a = \frac{40 * 4.454}{40.8} = 4.367 \text{ in}$$

$$M_{n_{40}} = \frac{40 * 4.454}{12} = (10 - 4.367/2) = 116.05 \text{ ft kips}$$

$$c = 4.367/0.85 = 5.138 \text{ in}$$

$$\varepsilon_s = 0.003\left(\frac{10 - 5.138}{5.138}\right) = 0.00284, \text{ compare with examples 1A and 2A.}$$

$$\varepsilon_y = 40/29000 = 0.00138$$

$$\varepsilon_s/\varepsilon_y = 0.00284/0.00138 = 2.06, \text{ compare with examples 1A and 2A.}$$

Example 3B

Same as example 3A except A_s given as 4.454 sq. in. and Grade 50 steel was supplied.

Find: ρ, $\bar{\rho}_b$, M_n, and ε_s

Thus:

$$\rho = 4.454/120 = 0.03712$$

$$\bar{\rho}_b = 0.03671$$

$\rho > \bar{\rho}_b$ $\therefore$ design would not be acceptable for any environment class if design were based on $f_y = 50$ ksi; brittle failure, design not permitted by ACI.

Since $\rho > \bar{\rho}_b$, then at ultimate moment:

$$\varepsilon_c = \varepsilon_u = 0.003, \text{ and}$$

$$(\varepsilon_s = f_s/E) < (f_y/E)$$

Now:

$$c = d\left(\frac{87}{87 + f_s}\right) \text{ and } a = \beta_1 c$$

so

$$a = 0.85\ d\left(\frac{87}{87 + f_s}\right)$$

also

$$a = \frac{f_s A_s}{0.85 f'_c b} = \frac{4.454 f_s}{40.8}$$

thus

$$8.5\left(\frac{87}{87 + f_s}\right) = \frac{4.454 f_s}{40.8}$$

solving

$$f_s = 49.59 \text{ ksi}$$

so

$$a = \frac{49.59 * 4.454}{40.8} = 5.41 \text{ in}$$

$$M_{n_{50}} = \frac{49.59 * 4.454}{12}(10 - 5.41/2) = 134.24 \text{ ft kips}$$

$$c = 5.41/0.85 = 6.365$$

$$\varepsilon_s = 0.003\left(\frac{10 - 6.365}{6.365}\right) = 0.00171$$

$$\varepsilon_y = 50/29000 = 0.00172$$

$$\varepsilon_s/\varepsilon_y = \frac{0.00171}{0.00172} = 0.993$$

This shows how moment capacity, etc. can be evaluated for brittle failure conditions.

The ratio of flexural strengths for beams of examples 3A and 3B are

$$\frac{M_{n_{50}}}{M_{n_{40}}} = \frac{134.24}{116.05} = 1.16$$

whereas the ratio of the yield strengths is

$$\frac{f_{y_{50}}}{f_{y_{40}}} = \frac{50}{40} = 1.25$$

Example 3C

Same as example 3A except A_s given as 4.454 sq. in. and Grade 60 steel was supplied.

Find: ρ, $\bar{\rho}_b$, M_n, and ε_s

Thus

$$\rho = 4.454/120 = 0.03712$$

$$\bar{\rho}_b = 0.02851$$

$\rho > \bar{\rho}_b$ $\therefore$ design would not be acceptable for any environment class if design were based on $f_y = 60$ ksi; brittle failure, design not permitted by ACI.

Again, at ultimate moment:

$$\varepsilon_c = \varepsilon_u = 0.003$$

$$f_s < f_y$$

so

$$a = 0.85 * 10\left(\frac{87}{87 + f_s}\right) = \frac{4.454 * f_s}{40.8}$$

from which, again

$$f_s = 49.59 \text{ ksi}$$

so

$$a = 5.41 \text{ in}$$

$$M_{n_{60}} = 134.24 \text{ ft kips}$$

$$\varepsilon_s = 0.00171$$

$$\varepsilon_y = 60/29000 = 0.00207$$

$$\varepsilon_s/\varepsilon_y = 0.00171/0.00207 = 0.827$$

Note:

$$\frac{M_{n_{60}}}{M_{n_{40}}} = \frac{134.24}{116.05} = 1.16$$

whereas

$$\frac{f_{y_{60}}}{f_{y_{40}}} = \frac{60}{40} = 1.50$$

Examples 3A, 3B, and 3C deal with other structures — with controlled environment with a design steel ratio $\rho = 0.75\bar{\rho}_{b_{40}}$. The examples illustrate that with a high design steel ratio, substitution of higher strength steels can cause the failure mode to switch from ductile to brittle with flexure strength increases that may be significantly less than the ratio of the yield strengths. The profession consensus is that brittle flexural failures are undesirable and should be avoided.

Alternate viewpoint

It might be helpful for insight to collect and discuss the results of the preceding examples in an alternate way. Instead of treatment in terms of environment class, the presentation could be made in terms of the design steel ratio versus the balanced steel ratio for the higher strength steel. Thus consider that Grade 40 steel is assumed in design, but a higher strength steel is supplied for construction. Now, if the design ρ is less than $\bar{\rho}_b$ for the higher strength steel, then (see examples 1A, 1B, 1C, 2A, 2B, 2C):

1. flexural strength is increased, but not as rapidly as the yield strength,
2. rotation capacity and ductility are decreased,
3. if moment redistribution was assumed in design, the load carrying capacity may not keep up with the increase in flexural strength, and
4. failure mode is ductile, but may not be sufficiently far from a brittle failure.

If the design ρ is more than $\bar{\rho}_b$ for the higher strength steel, then (see examples 3A, 3B, 3C):

1. flexural strength is increased relatively little,
2. rotation capacity is decreased, section is without ductility,
3. moment redistribution is not possible, and
4. failure mode is brittle and hence unacceptable.

Table 3 indicates that the ratio of flexural strengths is more significant than the ratio of yield strengths.

Table 3
Comparison of Ratio of Yield Strengths to
Ratio of Flexural Strengths

Example	ρ	f_y	$f_y/f_{y_{40}}$	$M_n/M_{n_{40}}$
1A	$\rho = \rho_{shy_{40}}$	40	1.00	1.00
1B	$\rho < 0.50\overline{\rho}_b$	50	1.25	1.22
1C	$\rho < 0.75\overline{\rho}_b$	60	1.50	1.42
2A	$\rho = 0.50\overline{\rho}_{b_{40}}$	40	1.00	1.00
2B	$\rho < 0.75\overline{\rho}_b$	50	1.25	1.20
2C*	$\rho < 1.00\overline{\rho}_b$	60	1.50	1.37
3A	$\rho = 0.75\overline{\rho}_{b_{40}}$	40	1.00	1.00
3B**	$\rho > 1.00\overline{\rho}_b$	50	1.25	1.16
3C**	$\rho > 1.00\overline{\rho}_b$	60	1.50	1.16

*insufficient ductility by ACI
**brittle failure

CONCLUSIONS, PRECAUTIONS, AND RECOMMENDATIONS

From the preceding investigations, example problems, and observations — the following conclusions, precautions, and/or recommendations seem rational with respect to the question of permissibility of substituting, during construction, higher strength reinforcing steel than was assumed in design.

1. Design provisions attempt to ensure ductile behavior at ultimate load. Changes to the design should not abrogate this goal.

2. The original design, for the lower strength steel, must be adequate in all respects, satisfying all requirements for the lower strength steel.

3. Permission to substitute should not be based on the presumption that a higher yield strength steel is an improvement. Both rotation capacity and ductility decrease with an increase in yield strength. The load carrying capacity of the system does not vary solely with steel yield strength.

4. Attempts at generalization may be risky. Design specific investigation is sometimes warranted.

5. As a guide, if moment redistribution was employed in design, the design steel ratio, ρ, should not be more than $0.5\overline{\rho}_b$ for the higher strength steel. If moment redistribution was employed in design and $\rho > 0.5\overline{\rho}_b$, then the load carrying capacity of the system may: be increased only slightly, remain essentially the same, or even be decreased.

6. As a guide, if moment redistribution was not employed in design, the design steel ratio, ρ, should not be more than $0.75\overline{\rho}_b$ for the higher strength steel. Sections with steel ratios exceeding this limit do not ensure sufficient ductility. Brittle failure is undesirable and should be avoided by a sufficient margin. Further, if substitution is made, the flexural strength ratio can be significantly less than the yield strength ratio.

7. Service hydraulic structures are designed for $\rho \leq \rho_{shy_{40}}$. These designs permit the substitution of higher strength steels. However, moment redistribution should not be employed in design if it is the designer's intent to permit the substitution of higher strength steels.

8. The grade of steel assumed in design should be clearly indicated in the drawings or specifications. If substitution of higher strength steels is not permissible, it should be so stated.

9. The validity of statements that the flexural strength of a section is increased by substitution of a higher strength steel than was assumed in design is totally dependent on the existence of adequate development of the higher strength steel. Thus if there is any need or intent to depend on an increased strength, then development lengths, standard hooks, and splices will need review and redesign at critical locations.

10. Discussion thus far has not mentioned sections or systems designed by working stress design. A reinforced concrete structure does not know by which method it was designed, i.e., strength design or working stress design. Hence all previous relations, etc. apply. It should be recognized that the use of the moment redistribution concept is usually not permitted in working stress design. Thus the design steel ratio, ρ, usually need only be checked against $0.75\overline{\rho}_b$ of the higher strength steel.

    Design steel ratios for working stress design are usually relatively low. In fact the maximum design steel ratios for working stress design and strength design for Service hydraulic structures are the same, i.e., $\rho_{max} = \rho_{bwsd} = \rho_{shy_{40}}$.

    These low steel ratios explain why, in working stress design, substitution of steels of higher strength than was assumed in design has not been perceived as a possible problem.

11. Finally, the question of substituting higher strength steel than was assumed in design is correctly of greater concern and significance in strength design than in working stress design precisely because of the higher steel ratios sometimes permitted in strength design.

REFERENCES

SCS:

NEH-6, Structural Design, Subsection 4. Reinforced Concrete
NEH Notice 6-4, Update of working stress design method
TR-67, Reinforced Concrete Strength Design
PS-313, Waste Storage Structure
NEM §536.20, Design criteria for reinforced concrete

Non-SCS:

ACI 318-77, Building Code Requirements for Reinforced Concrete
ACI, Commentary on Building Code Requirements for Reinforced Concrete (ACI 318-77)
PCA, Notes on ACI 318-77 Building Code Requirements for Reinforced Concrete