

Reference Manual for RC (Reinforced Construction) design using G-500 reinforcing Steel bars

For Practicing Engineers Based on ACI Code 318-11



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First Edition

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Foreword

“Reference Manual for RC Design using G-500 Reinforcing Steel” is neither a marketing manual nor a research digest. It is a courageous endeavor of a young Lady Consulting Engineer whose urge to share her experience with other structural engineers led to the development of this document.

Ethically a consulting engineer would never specify a particular brand and keeping the same spirit, Engr. Sadaf Fatima ably and strictly remained in her domain and her prime focus of facilitating structural engineers while using G-500 steel never experienced even a slight shift. The manual therefore, fundamentally serves as a ready reckoner to the structural engineering fraternity and facilitates them to directly apply the ACI Code Clauses without referring it back and forth.

It was this noble urge and sincere desire of Engr. Sadaf Fatima that encouraged me to review this document on her request. I am sure that this pleasant surprise which unfortunately has not yet been witnessed (with one or two exceptions, of course) would readily be welcomed by the structural engineering fraternity, and if at all, some areas are found where difference of opinion emerges, they could well be discussed with the lady, opening new appreciative avenues of desirable and rational improvements. This is how the developed world keeps on improving their documents through consultative process, ultimately reaching a consensus document.

Dr. Sahibzada Farooq Ahmad Rafeeqi
13th February, 2016.

Preface

Since times immemorial, human desire to strive for perfection has led to many advances in almost every field and Structural Engineering is no different. It has always been a challenge to arrive at an optimum structural design, which should not only fulfill the requirements of strength, serviceability, durability, functionality, and economy, but should also be aesthetically attractive and spacious. More space can be achieved by having greater spans with smaller dimensions of structural members and since architects would like to make use of every inch of space available, they keep asking structural engineers to push their design to minimal sizes. Engineers, therefore, are compelled to search for high strength and high performance materials which are not only easy to handle, but also possess the desired requirements and durability for a given loading and environmental regime. The same practices are prevalent in Pakistan.

Reinforced Concrete Construction is by far the most common practice in Pakistan. Concrete is a manufactured particulate material, which is inherently brittle in nature. Hence reinforcing steel bars are used with concrete to provide the desired level of ductility. This combination has proved to give a wonderful material, which is strong, ductile, and suitable for almost all kinds of structures.

For high-rise buildings made of reinforced concrete, steel placement during construction assumes an important role. Congestion of reinforcement at beam column joints, lap areas, shear walls, transfer girders, and other heavily reinforced sections makes concreting a big challenge and sometimes it becomes difficult to handle. To avoid this congestion, high strength steel reinforcing bars can be one of the alternates. With high strength steel, the required area would be less, leading to lesser number of reinforcing bars, making placement of concrete much easier. Occasionally though, the detailing requirements may offset the effort.

High strength Steel reinforcement bars having yield strength of 72.5 KSI (G-500 bars having 500 MPa yield strength) is being produced by a few manufacturers in our country and are being used by many engineers in mega structures.

Most of the structural engineering practitioners in Pakistan are following ACI Code 318 and reinforcing bars of yield strength of 60 KSI are in common use in the local Pakistani market, therefore, most of the Structural Engineers are familiar with the relative design requirements. Use of G500 bars, however, requires harmonization with the requirements of ACI Building Code, and thus, necessitates guidelines for use of G-500 steel by structural engineers, most of whom are familiar with ACI Code.

The basic objective of this Reference Manual, therefore, is to facilitate those structural engineers who choose to use G500 steel using guidelines given in ACI Code 318-11. Specific design requirements for reinforcing steel having yield strength of 72.5 KSI (500 MPa) as per ACI Code 318-11 are provided in this manual, such as, minimum thickness of structural members, minimum reinforcement, development lengths and lap lengths etc.

Furthermore, only for pertinent information for the user, a testing program was also conducted for Xtreme® G500 rebars produced by M/S Amreli Steels Limited for possible conformity of bars with BS standards. Three samples of each bar's diameter consisting of 10 mm, 12 mm, 16 mm, 20 mm and 25mm were tested for tensile properties and chemical composition test was performed on a sample of 10 mm diameter bar. Appendix A contains the reports of test results done by ACTS® lab, Qatar. Appendix B contains an example illustrating the design procedure of a beam and slab for a better understanding of the design process using G-500 reinforcement. The bar reckoner of M/S Amreli Steels Limited is attached in Appendix C.

Acknowledgements

M/S Amreli Steels Limited has sponsored the work and provided the technical information related to Xtreme® G-500 bars.

Special recognition is due for Dr. Sahibzada Farooq Ahmad Rafeeqi, (Retired Meritorious Professor & Ex-Pro VC-II, NED University of Engineering and Technology, Karachi) for reviewing this document and, whose continued guidance has made the production of this reference manual possible.

I would also like to extend my gratitude to my associate engineers, Muhammad Shoaib, who contributed immensely in developing Chapter 4 and 5 of this manual, Ms. Hira Rustam and Ms. Syeda Batool Zehra who were the key contributors in developing Chapter 3 of this manual.

This manual is developed for the users of G-500 steel reinforcement. This manual does not recommend any particular brand of steel. Selection of steel is solely on the discretion of Engineer.

The user must determine the applicability of all limitations/guidelines given in this document before applying it and must comply with all applicable governing laws and regulations.

Author disclaims liability for damages of any kind, including any special, indirect, incidental or consequential, including without limitation, lost revenues or lost profits, which may result from the use of this document.

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Chapter 1

Introduction

1.1 Why this Manual is needed?

Ultimate Strength Design method has been able to achieve lesser cross-sectional dimensions, however, it has also led to more congestion. Therefore, in concrete industry, there is a growing interest for the use of high strength reinforcing steel, which provides two-fold benefits by way of achieving congestion relief and attaining lesser amount of steel area for a given cross-section. In Pakistan, ACI Code is not only the commonly used Code for design practice, but Pakistan Building Code also refers to ACI Code and UBC 97 for Design, Detailing, and Seismic parameters. ACI Code, however, restricts use of reinforcing steel bars having yield strength up to 80 KSI for flexure and compression, yield strength up to 60 KSI for shear design and 100 KSI for confinement purpose. In Pakistan, Grade 60 steel reinforcement is in common use mainly due to unavailability of higher-grade steel.

In Pakistan, a few steel manufacturers are producing high strength steel bars having yield strength of 72.5 KSI (500 MPa) and this steel has started being used in many mega structures.

Almost all structural engineers in Pakistan are familiar with ACI code requirements and prefer to follow the same. There was a need to extract all the ACI code clauses, which account for higher rebar yield strength, and customize them for G-500, reinforcing steel by modifying the formulae, tables and/or graphs therein. This was done to facilitate the Structural Engineering Community, which is using Grade 60 reinforcement for designing of RCC structures, and is familiar with the Code requirements for grade 60 steel rebars. The Author herself being a Structural Engineer had to undergo repetitive modifications in the formulae and table while using G-500 reinforcing bars and thought it better to develop a ready reckoner to avoid such repetition for each design exercise. This reckoner will facilitate the structural engineering community and relieve them of this repetitive exercise. By referring to the Reference Manual for G-500 steel, a Structural Engineer will get all the relevant design data and will not have to search for code clauses regarding G-500 reinforcing bars and recalculate them for yield strength of 72.5 KSI (500 MPa).

1.2 Areas covered in Reference Manual

Reinforcement characteristics as outlined by BS code are remainder of this first Chapter while Chapter 2 presents notations used in this document. Chapter 3 discusses in detail the use of G-500 steel and its related issues. Chapter 4 contains design aids, which are developed and structured from ACI code 318-11. References are given from the Code with each Section. Solved formulae for various values are tabulated for ready reference and where more than one variable is involved, only the formula is noted down. Chapter 5 describes limitations on use of high strength steel based on ACI code 318-11.

A testing program was conducted to give the user information related to G-500 reinforcement, in which 15 samples of Xtreme® G-500 bars produced by M/S Amreli Steels Limited were tested at a recognized international lab, ACTS, Qatar. Test results are attached in Appendix-A of this report. Appendix-B demonstrates RC design procedure using G-500 rebar with a solved example. Appendix C has the bar reckoner of M/S Amreli Steels Limited attached for reference.

1.3 Reinforcement characteristics as per British Code

Following are the characteristic tensile properties of reinforcement as per British standard BS 4449-05:

Yield Strength, R_e (MPa)	500
Tensile to Yield Strength ratio, R_m/R_e	≥ 1.15 , < 1.35
Total Elongation at max. Force, A_{gt} (%)	7.5
Rebend Test Dia (d = Nominal dia. of reinforcing steel)	4d (≤ 16 mm \emptyset bar), 7d (> 16 mm \emptyset bar)
Carbon Equivalent (by product analysis)	0.52 (max. % by mass)

Results of testing of Xtreme® G-500 bars produced by M/S Amreli Steels are compared with BS- 4449 code limits, and tabulated in Section 3.4.2 of this report.

Chapter 2

Notations

α_{fm}	= average value of α_f for all beams on edges of a panel
α_f	= ratio of flexural stiffness of beam section to flexural stiffness of a width of slab bounded laterally by centerlines of adjacent panels (if any) on each side of the beam
Ψ_e	= factor used to modify development length based on reinforcement coating
Ψ_t	= factor used to modify development length based on reinforcement location
λ	= modification factor reflecting the reduced mechanical properties of lightweight concrete, all relative to normal-weight concrete of the same compressive strength
ρ_s	= ratio of volume of spiral reinforcement to total volume of core confined by the spiral (measured out-to-out of spirals)
A_{ch}	= cross-sectional area of a structural member measured to the outside edges of transverse reinforcement, in ² ,
A_g	= Gross area of concrete section, in ² . For a hollow section, A_g is the area of the concrete only and does not include the area of the void(s),
A_s	= area of non-prestressed longitudinal tension reinforcement, in ² ,
$A_{s\ min}$	= minimum area of flexural reinforcement, in ² ,
A	= area of concrete surrounding each bar, in ² ,
b_w	= web width, or diameter of circular section, in,
C_c	= clear cover of reinforcement, in,
C_b	= distance from cover till C.G of steel bar, in,
d	= distance from extreme compression fiber to centroid of longitudinal tension reinforcement, in,
d_b	= nominal diameter of bar, wire, or pre-stressing strand, in.
d_c	= thickness of concrete calculated from extreme tension fiber to the center of bar, in,
a	= depth of equivalent rectangular stress block as defined in ACI 318-11 Chap 10, in,
f'_c	= specified compressive strength of concrete in PSI,
f_s	= calculated tensile stress in reinforcement at service loads, in PSI
f_y	= specified yield strength of reinforcement, in PSI,
f_{yt}	= specified yield strength f_y of transverse reinforcement, in PSI,
h	= overall thickness or height of member, in,
l	= span length of beam or one-way slab; clear projection of cantilever, in,
l_d	= development length in tension of deformed bar, deformed wire, plain and deformed welded wire reinforcement, or pre-tensioned strand, in,

l_{dc}	= development length in compression of deformed bars and deformed wire, in,
l_{dh}	= development length in tension of deformed bar or deformed wire with a standard hook, measured from critical section to outside end of hook (straight embedment length between critical section and start of hook [point of tangency] plus inside radius of bend and one bar diameter), in,
l_n	= length of clear span measured face-to-face of supports, in,
N_c	= tension force in concrete due to un-factored dead load plus live load, lb.,
s	= center-to-center spacing of items, such as longitudinal reinforcement, transverse reinforcement, pre-stressing tendons, wires, or anchors, in,
ϵ_t	= net tensile strain in extreme layer of longitudinal tension steel at nominal strength, excluding strains due to effective pre-stress, creep, shrinkage, and temperature.
ϵ_s	= ultimate tensile strain
M_u	= Factored moment at section, in-lb.,
M_n	= nominal flexural strength at section, in-lb.,
ϕ	= curvature of cross-section
ϕ_u	= curvature of cross-section at the end of post-elastic range
ϕ_y	= curvature of cross-section at first yield
M	= curvature ductility ratio
R_u	= a factor that depend on the steel ratio ρ , PSI
P	= ratio of A_s to bd
ρ'	= ratio of A_s' to bd
W	= crack width, in ,

Chapter 3

General Discussion on use of G-500 Bars

High strength materials are the need of this age. With advancement in every field, we are continuously searching for structural components that are lighter and stronger, improve energy efficiencies, reduce emissions & pollution, increase safety, and, cost less to produce.

The compatible strength of both concrete and steel is of vital importance due to many good reasons mostly known to the structural engineers. Therefore, while using high strength steel, the designer has to be careful about selecting the combination of Ultimate Concrete Compressive Strength. Some of the main reasons as discussed in Section 3.1 of this manual led to the recommendation that low strength concrete should not be used with high strength steel, and high strength concrete should not be used with low strength steel.

3.1. Combination of Concrete Strength and Steel Strength

Use of high strength concrete with low strength steel results in crowding of steel reinforcement and hence, a costly cross-section, as maximum. Steel percentage ratio (ρ_{max}) for the cross-section comes out to be higher. For the same case, if low percentage of steel is used, concrete may not be utilized economically. Whereas, if low strength concrete is used with high strength steel, maximum allowable steel percentage comes out to be very low and thus reduces the stiffness of member, causing excessive deflection and cracking.

As per discussion in Section 4.6.2 of “Design of Reinforced Concrete Structures” by Nadeem Hassoun, [6], range of f_y/f'_c between 12 and 16 is considered most practical. However, in Pakistan, 3 KSI cylinder concrete strength with G-60 steel is being used and it has shown satisfactory performance, deducing therefore, that a maximum value of f_y/f'_c of 20 may be fine. Table 3.1 shows different ρ_{max} for different values of f_y/f'_c . It also shows that concrete strength of minimum 4000 PSI is appropriate in combination with G-500 steel to remain in the desirable limits of f_y/f'_c .

Table 3.1:

f'_c/f_y	3000PSI		3500PSI		4000PSI		4500PSI		5000PSI		6000PSI	
	ρ_{max}	f_y/f'_c	ρ_{max}	f_y/f'_c	ρ_{max}	f_y/f'_c	ρ_{max}	f_y/f'_c	ρ_{max}	f_y/f'_c	ρ_{max}	f_y/f'_c
60	1.60	20	1.87	17.14	2.14	15	2.33	13.33	2.52	12	2.83	10
72.5	1.22	24.17	1.43	20.71	1.63	18.13	1.78	16.11	1.92	14.5	2.16	12.08

3.2. Design Considerations

Reinforced Concrete design for seismic area structures is done in three stages: strength, serviceability and ductility.

Design for strength involves load magnification factor, which ensures adequate safety against an increase in service loads, beyond loads specified in design, so that failure is unlikely to occur. The strength reduction factors allow for approximations in the calculations and variations in material strengths, workmanship, and dimensions.

Members with smaller cross-section and percentage of compression steel may satisfy the strength requirement, but the section may experience high stresses that may lead to unacceptable deformations at service load. Larger deformations lead to wider cracks which may be undesirable from both aesthetic and durability point of views. Hence, serviceability check for structural performance is critical. A fundamental issue in using G-500 or any other high strength reinforcing steel is that the stress at service load (f_s assumed to be about $0.6f_y$) is expected to be greater in comparison to that of conventional Grade-60 steel. Consequently, the service load reinforcing strains (i.e., $\epsilon_s = f_s/E_s$) shall be greater than those for conventional Grade-60 steel.

Another significant consideration in reinforced concrete seismic design is to ensure that in the extreme event of structure being loaded to failure, it should behave in a ductile manner by giving ample warning at near-maximum load carrying capacity because the ductility of a member decreases with increase in yield strength of steel. Therefore, the use of G-500 necessitates due consideration regarding ductility.

Serviceability and Ductility of structures built with high strength steel are discussed in the following Sections:

3.2.1. Serviceability

Performance of structures at service loads is an important design consideration. If cross-sections are proportioned by strength requirements alone,

there is a risk, although the degree of safety against the collapse will be adequate, but the performance of structure at service loads may be unsatisfactory.

Hence, the structure should be designed with reference to service limits with the most important being strength at ultimate load deflection at service loads and crack widths at service loads.

Deflection Control

With use of concrete and steel of higher strengths, the introduction of strength design of more slender structural elements has become possible. Non-structural members in modern building structures may be prone to damage caused by deformation of structural members. Hence, control of deflections of flexural members assumes greater importance.

Deflections may be controlled by ensuring that members have sufficient stiffness to limit the deformations at the service loads. ACI Code has two methods for controlling deflections:

- **Use of Limiting Span/Thickness Ratios:**

- For beams and one-way slabs, the deflection requirements may be considered satisfactory, if the minimum overall thickness is not less than those specified in Table 9.5(a), ACI Code 318-11. See Section 4.4.1 of this manual for converted values for G-500 steel
- Minimum thickness for two-way slabs without interior beams should be as per Table 9.5(c), ACI Code 318-11. See Section 4.4.2 of this manual for converted values for G-500 steel
- For beams, one-way slabs and two way slabs, which do not meet the requirements of Table 9.5(a) and Table 9.5(c) of ACI Code, deflections must be calculated and are limited to the values listed in Table 9.5(b) of ACI Code 318-11

- **Control of Cracking**

The occurrence of cracks in reinforced concrete structures is inevitable because of low tensile strength of concrete. Structures designed with low steel stresses at the service load serve their intended function with limited cracking. However, with high service load stresses, particularly because of the use of high-strength steel, some cracking is expected at the service load. The cracking of a reinforced structure at the service load should not be such to spoil the appearance of

the structure and/or to lead to other ductility problems related to both concrete and steel.

“The maximum size of a crack that may be considered non-detrimental to the appearance of a member, or non-conductive to the feelings of alarm, depends on the position, length, width illumination, and surface texture of the crack. The maximum crack width that will neither impair a structure’s appearance nor create public alarm is probably in the range of 0.01 to 0.015inch (0.25 to 0.38mm) but larger crack widths may be tolerated.

At present, cracking is controlled by specifying maximum allowable crack widths at the surface of the concrete for given types of environment”. [4]

The permissible values for width of cracks as recommended by ACI committee 224, are listed in following Table (Table 4.1, ACI 224R-01)

Table 3.2:

Exposure Condition	Crack Width	
	In.	Mm
Dry air, or protective membrane	0.016	0.41
Humidity, moist air, soil	0.012	0.30
Deicing chemicals	0.007	0.18
Seawater and seawater spray, wetting and drying	0.006	0.15
Water-retaining structures*	0.004	0.10

Note: It should be expected that a portion of the cracks in the structure would exceed these values. With time, a significant portion can exceed these values. These are general guidelines for design to be used in conjunction with sound engineering judgment.

3.2.2. Ductility

Ductility may be defined as the ability to undergo deformations without a considerable reduction in the flexural capacity of the member. This deformability in reinforced concrete structures is influenced by some factors such as the tensile reinforcement ratio (ρ), ratio of compression reinforcement to tensile reinforcement (ρ' / ρ), the amount of lateral tie and, the strength of concrete (f'_c). With reference to Section 6.3.1, “Reinforced Concrete Structures” by R. Park & T. Pauly [3], following are the factors contributing to the ductility of reinforced concrete member, keeping the area of steel and cross section of member constant.

- An increase in the steel yield strength decreases the ductility
- An increase in concrete strength increases the ductility
- An increase in compression steel content increases the ductility
- An increase in the tension steel content decreases the ductility
- An increase in extreme fiber concrete strain at ultimate increases the ductility

Ductile reinforced structure frame requires capability of undergoing large deflections at near-maximum load carrying capacity, that may save lives by giving warning of failure and prevent total collapse. This overall ductility of the structural system is achieved by possible redistribution of bending moments, shear force, and axial load that depends on ductility of the members at critical sections.

How this redistribution is guaranteed: As ultimate load is approached, some cross-sections may reach their ultimate resisting moments before others, and if cross section possesses the ability of plastic rotation, while the ultimate moment is maintained, additional load can be carried as the moment elsewhere increases to their ultimate value. The ultimate load of structure is reached when, after the formation of sufficient plastic hinges, a collapse mechanism is developed, thus ensuring higher load carrying capacity.

How ACI Code is ensuring ductility: In earlier ACI Codes, for example Code of 1971 and 1977, ductility was ensured in semi-quantifiable manner by setting limits on longitudinal steel reinforcement ratios in a cross-section of member as follows, and by providing confinement reinforcement:

- $(\rho - 0.75\rho') \leq 0.75\rho_b$, limit for longitudinal reinforcement in cross-sections for general cases, i.e., $\rho < 0.75\rho_b$ for a case of singly reinforced section
- $(\rho - \rho') \leq 0.5\rho_b$, limit for longitudinal reinforcement in cross-sections where redistribution is utilized and
- $(\rho - 0.5\rho') \leq 0.5\rho_b$, limit for cross-sections of structures in seismic areas

Present day, ACI Code, though do not explicitly define ductility in terms of reinforcement ratios, however takes care of the same by putting limits on net tensile strains in extreme layer of longitudinal tension steel (Refer Section 8.4 ACI Code 318-11) and through detailing requirements. Experimental studies show that members satisfying the Code requirements possess adequate rotation capacity for moment distribution allowed by ACI Code (Refer commentary R 8.4, ACI Code 318-11). The above statement is made keeping in mind that steel used is as per standards set by ASTM.

Although the ACI Code has not provided any guideline on using higher concrete strength value with higher grade steel, it has chosen to ensure the same effect by way of setting increased member depth requirements for high strength steels (refer to Section 9.5, ACI Code 318-11). Increased depth of member, in case of high strength steel on one hand, improves the ductility, and on the other, ensures lesser stresses in tensile steel, and results in controlled crack widths at ultimate, an effect comparable to increased compressive strength.

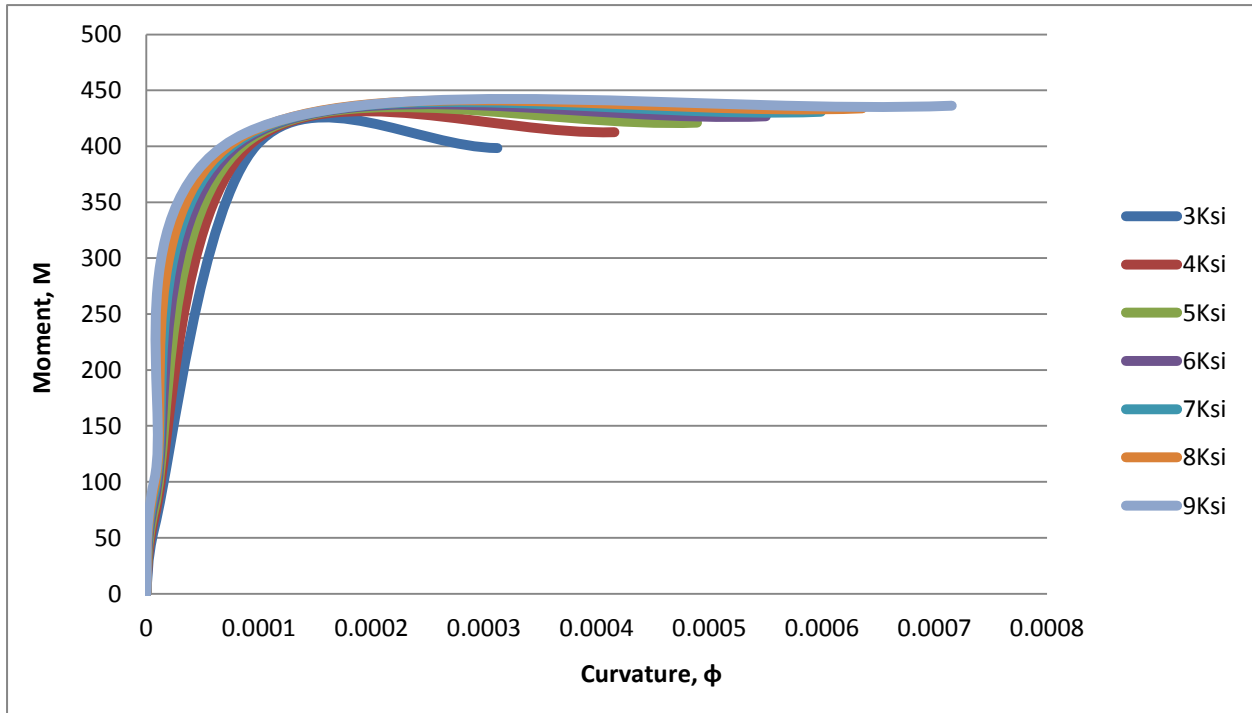
The ductility of a reinforced concrete section is usually expressed in terms of curvature ductility ratio ϕ_u/ϕ_y where ϕ_u is curvature at the end of the post-elastic range and ϕ_y is curvature at first yield. To elaborate the discussion above, solved examples have been given in the following sections:

- Example 1: Moment-curvature graphs are developed for a beam cross-section designed with G-60 steel with seven different values of f'_c .
- Example 2: Moment-curvature graphs are developed for same beam cross-section with same loading and boundary conditions as in Example 1, but designed with G-500 steel with seven different values of f'_c .
- Example 3: A beam with a constant width of 12inches, same boundary conditions and with different concrete strengths ranging from 4 to 6 KSI is designed for two types of loading, normal and heavy, each with 3 cases as follows:
 - With G-60 steel
 - With G-500 steel
 - With G-500 steel having same member depths as required for G-60 steel

3.2.2.1. Example 1- Beam with G-60 reinforcement

An interior beam of a residential structure having size 8" x 36" is considered for analysis using G-60 bar. Singly reinforced unconfined cross-section of the beam is used to evaluate ductility with different values of concrete strength (f'_c) having same reinforcement steel.

M- ϕ curves are generated for each case and curves' main parameters are tabulated.



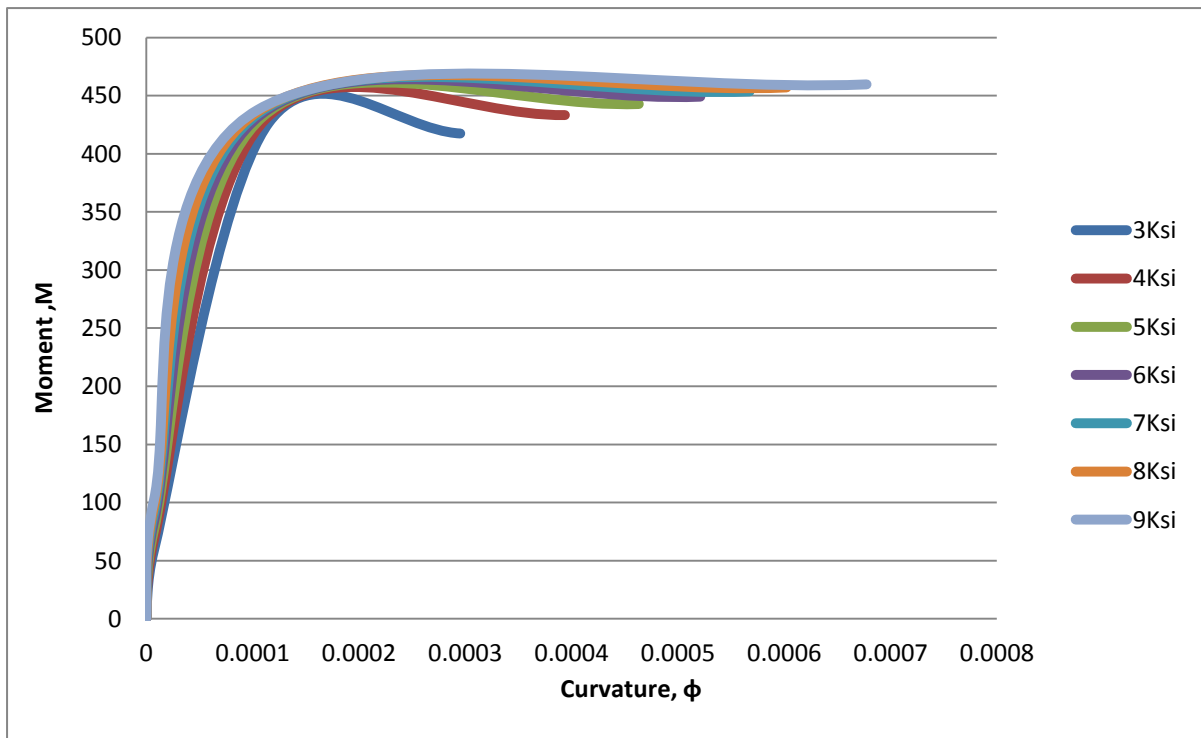
M- ϕ Curve for RCC beam with G-60 steel having different concrete strengths

f'_c (KSI)	M_u (k-ft.)	ϕ_u (rad/in)	ϕ_y (rad/in)	ϵ_s	$\phi_u/\phi_y = \mu$
3	398.40	0.000312	0.000098	0.005421	3.18
4	412.61	0.000416	0.000095	0.005914	4.37
5	421.13	0.000489	0.000093	0.006323	5.25
6	426.81	0.000550	0.000092	0.006674	6.01
7	430.87	0.000599	0.000090	0.006985	6.64
8	433.92	0.000636	0.000089	0.007264	7.12
9	436.28	0.000715	0.000088	0.007518	8.10

3.2.2.2. Example 2- Beam with G-500 reinforcement

Same beam as in previous example, with same loading and size is designed with Grade 500 steel and moment-curvature curve is developed for different values of f'_c .

M- ϕ curves are generated for each case and curves' main parameters are tabulated.



M- ϕ Curve for RCC beam with G-500 steel having different concrete strengths

f'_c (KSI)	M_u (k-ft.)	ϕ_u (rad/in)	ϕ_y (rad/in)	ϵ_s	$\phi_u/\phi_y = \mu$
3	417.42	0.0002953	0.000115274	0.00588126	2.56
4	433.27	0.0003937	0.000112064	0.00640971	3.51
5	442.77	0.0004632	0.000109779	0.00684761	4.22
6	449.11	0.0005211	0.000108034	0.0072246	4.82
7	453.64	0.0005674	0.000106638	0.00755744	5.32
8	457.04	0.0006021	0.000105484	0.00785661	5.71
9	459.68	0.0006774	0.000104507	0.00812913	6.48

3.2.2.3. Example 3- Design of Beam with G-60 and G-500 reinforcement as per ACI Code

A rectangular beam with a constant width of 12inches, same boundary conditions and with concrete strengths ranging from 4 to 6 KSI has been designed for two types of loading, normal and heavy, each with 3 cases as follows:

- 1- With Grade 60 steel
- 2- With G-500 steel
- 3- With G-500 steel, keeping depth of beam same as in case of G60 steel

Normal Loading:

Case-1: G60

f'_c	f_y/f'_c	d_{req}	$h_{prov.}$	ρ_{bal}	ρ_{max}	ρ_{req}	A_s	c/d_t	ϵ_t
4	15	14.61	19	0.0285	0.0213	0.0178	3.35	0.3715	0.005073
4.5	13.33	13.93	18.5	0.0311	0.0233	0.0188	3.42	0.3589	0.005356
5	12	13.38	18	0.0335	0.0251	0.0200	3.51	0.3536	0.005484
6	10	12.53	17	0.0377	0.0282	0.0229	3.74	0.3593	0.005348

Case-2: G-500

f'_c	f_y/f'_c	d_{req}	$h_{prov.}$	ρ_{bal}	ρ_{max}	ρ_{req}	A_s	c/d_t	ϵ_t
4	18.13	15.08	19.5	0.0217	0.0163	0.0137	2.65	0.3441	0.005717
4.5	16.11	14.39	19	0.0237	0.0178	0.0144	2.71	0.3320	0.006034
5	14.5	13.82	18.5	0.0255	0.0191	0.0153	2.77	0.3264	0.006188
6	12.08	12.95	18	0.0287	0.0215	0.0160	2.82	0.3046	0.006846

Case-3: G-500 (same cross-section as required for G-60)

f'_c	f_y/f'_c	d_{req}	$h_{prov.}$	ρ_{bal}	ρ_{max}	ρ_{req}	A_s	c/d_t	ϵ_t
4	18.13	15.083	19	0.0217	0.0163	0.0148	2.77	0.3715	0.005073
4.5	16.11	14.390	18.5	0.0237	0.0178	0.0156	2.83	0.3589	0.005356
5	14.5	13.821	18	0.0255	0.0191	0.0165	2.91	0.3536	0.005484
6	12.08	12.951	17	0.0287	0.0215	0.0189	3.09	0.3593	0.005348

Heavy Loading:

Case-1: G-60

f'_c	f_y/f'_c	d_{req}	$h_{prov.}$	ρ_{bal}	ρ_{max}	ρ_{req}	A_s	c/d_t	ϵ_t
4	15	25.25	29	0.0285	0.0213	0.02039	6.27	0.4235	0.004083
4.5	13.33	24.08	28	0.0311	0.0233	0.02186	6.46	0.4156	0.004217
5	12	23.12	27	0.0335	0.0251	0.02361	6.69	0.4167	0.004198
6	10	21.65	26	0.0377	0.0282	0.02518	6.83	0.3951	0.004592

Case-2: G-500

f'_c	f_y/f'_c	d_{req}	$h_{prov.}$	ρ_{bal}	ρ_{max}	ρ_{req}	A_s	c/d_t	ϵ_t
4	18.13	26.06	31	0.0217	0.0163	0.01400	4.64	0.3512	0.005540
4.5	16.11	24.86	30	0.0237	0.0178	0.01493	4.77	0.3431	0.005741
5	14.5	23.88	29	0.0255	0.0191	0.01603	4.93	0.3419	0.005772
6	12.08	22.38	28	0.0287	0.0215	0.01705	5.04	0.3233	0.006278

Case-3: G-500 (same cross-section as required for G-60)

f'_c	f_y/f'_c	d_{req}	$h_{prov.}$	ρ_{bal}	ρ_{max}	ρ_{req}	A_s	c/d_t	ϵ_t
4	18.13	26.06	29	0.0217	0.0163	0.01688	5.19	0.4235	0.004083
4.5	16.11	24.86	28	0.0237	0.0178	0.01809	5.34	0.4156	0.004217
5	14.5	23.87	27	0.0255	0.0191	0.01954	5.54	0.4167	0.004198
6	12.08	22.37	26	0.0287	0.0215	0.02084	5.65	0.3951	0.004592

3.3. Discussion on Example1, 2 and 3

When comparing moment curvature curves generated in Example 1 (with G-60 bars) and Example 2 (with Grade 500 bars), it is clear that as concrete strength increases, the ductility increases. Moreover, we can observe from the tabulated values of 'μ' that curvature ductility of reinforced concrete beam designed with Grade 60 beam having concrete strength of 3 KSI is achieved when Concrete strength of 4000 PSI is used with G-500 bars. Hence, it is advisable to use **minimum of 4,000-PSI cylinder strength concrete with G-500 bars.**

In Example 3, and normal condition case, when comparing Case 1 and Case 2, we see that while using G-500 steel:

- Depths required are more as compared to G-60 case
- The strains are more when G-500 steel is used
- Area of steel required decreases in Case 2 by 20 percent
- Members are tension controlled

Whereas, in Case 3, when depth of member is kept same as in case of G-60, we notice that:

- Area of steel increases as compared to Case 2 but still is less than that of Case 1
- Strain at ultimate level is same as that of Case-1

For Heavy loading condition, when comparing Case 1 and 2, we see that:

- Cross-section is in transition zone in Case 1 whereas, it is tension controlled in Case 2
- Depth required in Case 2 is more than in Case 1, and difference in depth requirement for heavy loading is more than in normal loading condition

Whereas, in Case 3, when depth of member is kept same as in case of G-60, we notice that:

- Strain in steel is same as in Case 1
- Area of steel required is more than that of Case 2 but still lesser than that of Case 1
- Cross-section is in transition zone

On an overall look, we notice that as concrete strength increases, required member depth decreases with increase in area of steel, and strains at ultimate level. Hence, we can deduce that, lesser depth can be kept with increased concrete strength for the member designed with G-500 steel, as compared to the values calculated from guidelines given in ACI Code. However, designer needs to check the serviceability requirements, as due to decrease in area of steel and increase in steel stress, crack widths will tend to increase.

3.4. Manufacturing process of Xtreme® G-500 Bars by Amreli Steels

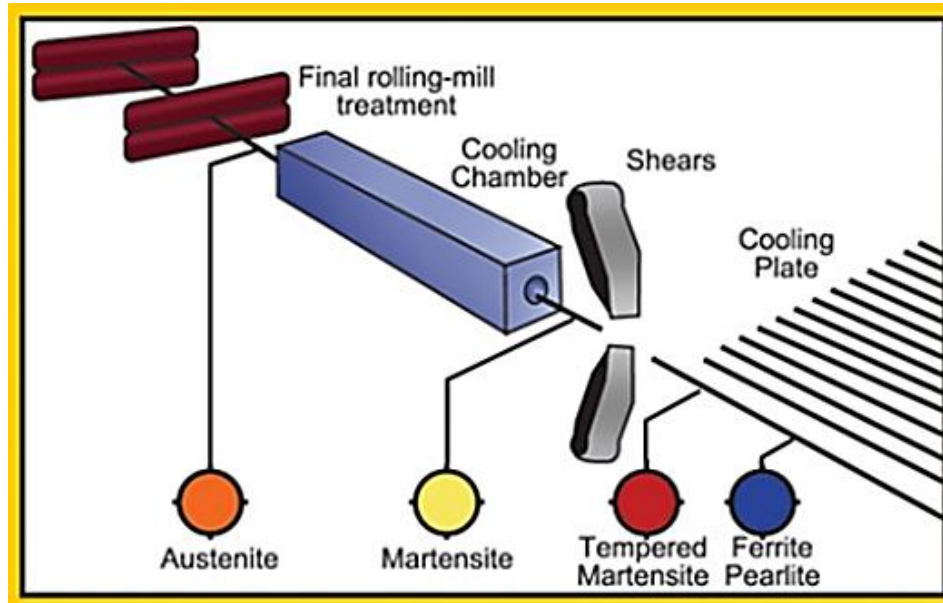
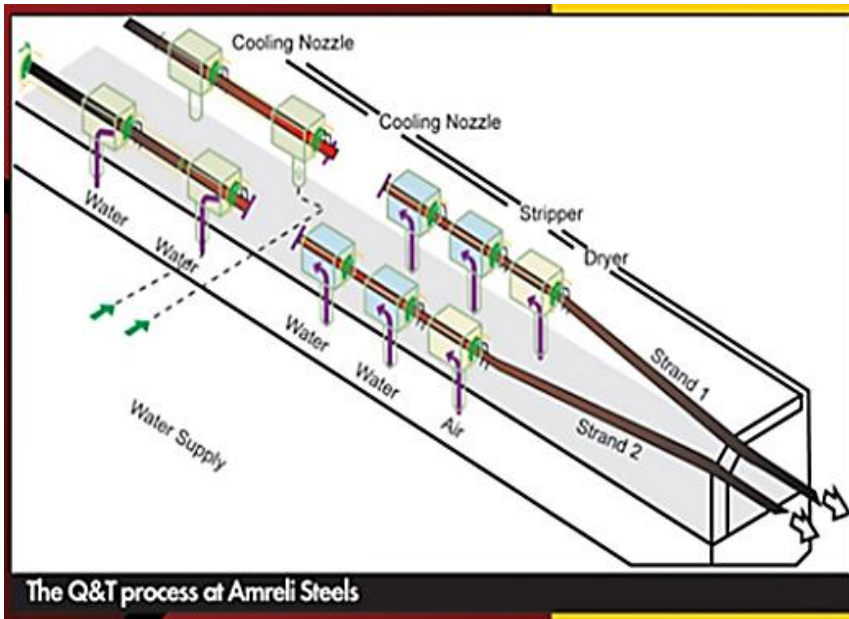
Anticipating that engineers may like to know the manufacturing process and engineering properties of Xtreme® G-500 bars produced by M/S Amreli Steels Limited, it was thought better to provide some details of manufacturing processes which are outlined in the following Section. A testing program was conducted in which Xtreme® G-500 bars' samples were sent to an ACI and ASTM recognized lab, M/S ACTS in Qatar for testing. Tensile testing was done on 15 samples and chemical test was done on one sample. Results are attached in Appendix-A of this report.



3.4.1. Manufacturing of Xtreme® G-500 Bars

In recent decades, dedicated efforts have been witnessed for the development of low-carbon advanced high strength steels possessing adequate ductility for many good reasons as discussed above and for reducing energy depletion, saving raw materials, as well as protecting environment. The reinforcing steel thus manufactured as an outcome product of such efforts is a combination of strength and elongation that indicates the balance of strength and ductility.

Quenching and tempering or Thermo-Mechanical Treatment is the process through which the steel bars attain hard surface and soft core to achieve high yield strength with ductility. Xtreme® G-500 rebars are hot-rolled from steel billets and subjected to online thermo-mechanical treatment in three successive stages as described below.



a) Quenching:

The hot-rolled bar leaving the mill stand is rapidly quenched by high-pressure water in special venture tubes. This hardens the surface of the bar to a depth optimized for each section through formation of martensitic rim while the core remains hot and austenitic.

b) Self-Tempering:

As the bar leaves the quenching venture tube, the core remains hot compared to the surface, allowing heat to flow from the core to the surface, which causes tempering of the outer martensitic layer into a structure, called tempered martensite. The core remains austenitic at this stage.

c) Atmospheric Cooling:

This takes place on the cooling bed, where the austenitic core is transformed into a ductile ferrite-pearlite structure. Thus, the final structure consists of an optimum combination of strong outer layer (tempered martensite) with a ductile core (Ferrite-pearlite). This provides a unique combination of high strength and ductility.

3.4.2. Properties and Characteristics of Xtreme® G-500

Xtreme® G-500			
	Amreli Steels Lab Results	ACTS Testing Results *	BS 4449-05 Requirements
Steel Grade	500	500	500
Manufacturing Process	Quenched and Tempered (Q & T Process) or Tempcore Process		
Key Characteristics	Outer Portion of the bar cross-section is harder and stronger than the ductile inner Portion		
Carbon Equivalent (%)		0.3799	0.52
Ultimate Tensile Strength	575Mpa	691Mpa	
Yield Strength	500Mpa	583Mpa	Max. Permissible value 650Mpa
Ratio of Tensile Strength to Yield Stress	1.15	1.185	$1.15 \leq x < 1.35$
Uniform Elongation (Measured at Maximum Force or after Fracture)	14% Minimum	15.5%	7.5%
Nominal Diameters	Dia in mm 10,12,16,20,25,32 & 40	Tested Dia in mm 10, 12,16,20,25	Nominal Dia in mm are 8,10,12,16,20,25,32 & 40
Mass	0.007847 kg/mm²/m	0.007796 kg/mm²/m	0.00785 kg/mm²/m

* Test results for 15 samples have been averaged out.

Chapter 4

Design Aids for G-500 Rebar

This Chapter is developed to facilitate the structural engineer who uses ACI Code for the reinforced concrete design. For G-500 steel, designer will have to do modification in equations of ACI Code every time. To omit this cyclic exercise to go back to Code again and again, there is a ready reckoner developed in this Chapter which will facilitate the designer by saving his/her time.

4.1. Guidelines to use the Design Aids for G-500 Rebar

- These Design Aids should be read in conjunction with ACI Code 318-11
- At the beginning of every section, reference to ACI Code 318-11 is given
- The Code clauses which are copied from ACI Code are written in *italic* in this chapter
- The tables of ACI Code modified for 72.5 KSI yield strength are numbered after the main section they are placed in and “m” is added at the end of table number. For example, in Section 4.4.2, table which is modified from table 9.5(a) of ACI Code 318-11 is numbered as 4.4.2m
- Serial numbers of equations mentioned in this Chapter are kept same as they appear in ACI Code 318-11

4.2. Temperature and shrinkage reinforcement in slabs

Reference: Section 7.12.2.1

“Area of shrinkage and temperature reinforcement shall provide at least the following ratios of reinforcement area to gross concrete area, but not less than 0.0014:

** Slabs where reinforcement with yield stress exceeding 60,000 PSI measured at a yield strain of 0.35 percent is used $0.0018 \times 60,000/f_y$ ”.*

Therefore, for $f_y = 72,500$ PSI, ratio of reinforcement area to gross concrete area = 0.00149.

4.3. Strength Reduction Factors

Reference: Section 9.3.2

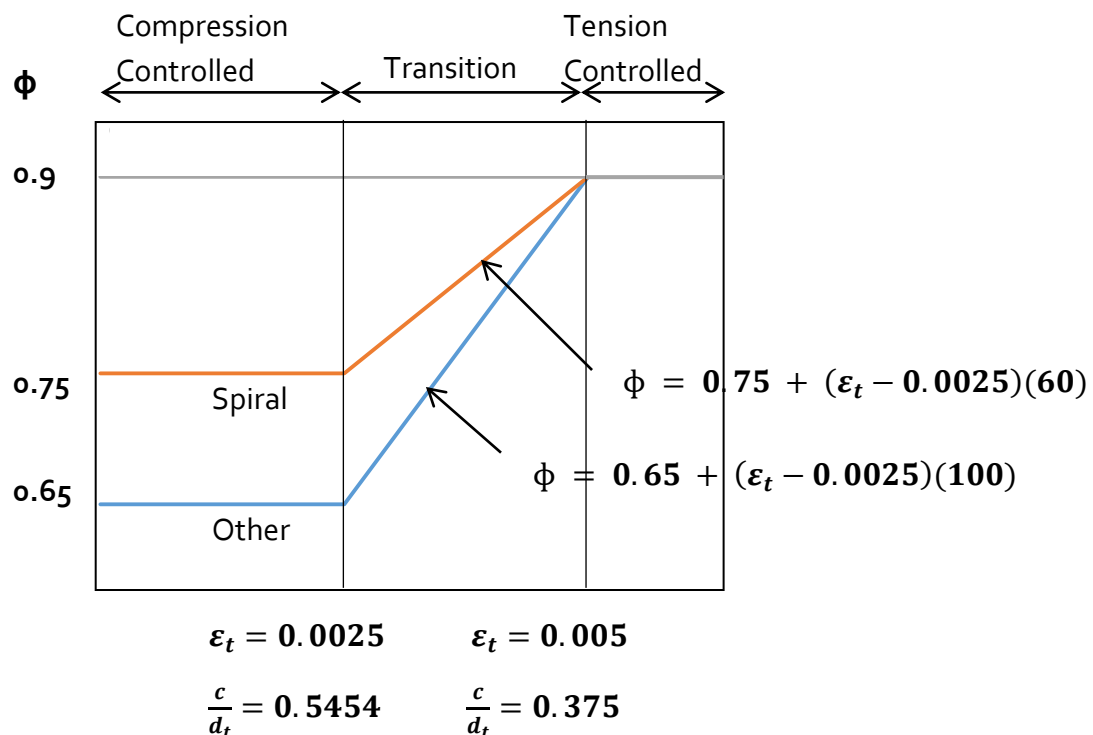
Strength reduction factors for design of a member have to be determined by strain conditions at a cross-section of member at nominal strength. As per ACI Code,

“Compression controlled and Tension controlled sections are defined in Section 10.3.3 and 10.3.4 as those that have net tensile strain in the extreme tension steel at nominal strength less than or equal to the compression controlled strain limit, and equal to or greater than 0.005, respectively.”

In Code Commentary R 10.3.4, it is stated that:

“The net tensile strain limit of 0.005 for tension-controlled sections was chosen to be a single value that applies to all types of steel (prestressed and non-prestressed) permitted by this Code”.

Below is the modified graph (with reference to ACI Code 318-11, Fig. R 9.3.2) for strength reduction factors. The starting strain at transition point is kept the same, i.e. $\epsilon_t = 0.005$ as specified by ACI. Changing the strain at start of compression control as $\epsilon_t = 0.0025$ (which is coming out due to use of G-500) modifies the slopes of the graph in the transition zone accordingly.



Interpolation on c/d_t : *spiral* $\phi = 0.75 + 0.18 \left[\left(\frac{1}{c/d_t} \right) - \left(\frac{11}{6} \right) \right]$

other $\phi = 0.65 + 0.30 \left[\left(\frac{1}{c/d_t} \right) - \left(\frac{11}{6} \right) \right]$

Modified relation of ϕ with net tensile strain in extreme tension steel, ϵ_t , and c/d_t , for Grade 500 reinforcement.

4.4. Minimum thickness of flexural members

4.4.1. Beams and One-way slabs

Reference: Section 9.5.2.1

Minimum thickness of non-prestressed beams or one-way slabs, unless deflections are calculated, is given in Table 4.4.1m.

Note: Following Table 4.4.1m is based on Table 9.5(a) of ACI Code 318-11.

Table 4.4.1m- MINIMUM THICKNESS OF NON-PRESTRESSED BEAMS OR ONE-WAY SLABS UNLESS DEFLECTIONS ARE CALCULATED

Member	Minimum thickness, h			
	Simply supported	One end continuous	Both ends continuous	Cantilever
	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections			
Solid one-way slabs	$l/17.8$	$l/21.3$	$l/24.9$	$l/8.9$
Beams or ribbed one-way slabs	$l/14.2$	$l/16.4$	$l/18.7$	$l/7.1$

4.4.2. Two way slabs without interior beams

Reference: Section 9.5.3.2

Minimum thickness of non-prestressed two-way slabs without interior beams shall be in accordance with Table 4.4.2m unless deflections are calculated, and shall not be less than:

*“4 inches for slab with drop panels;
5 inches for slab without drop panels.”*

Note: Following Table 4.4.2m is based on Table 9.5(c) of ACI Code 318-11.

TABLE 4.4.2m- MINIMUM THICKNESS OF SLABS WITHOUT INTERIOR BEAMS

f_y (PSI)	Without drop panels‡			With drop panels‡		
	Exterior Panels		Interior panels	Exterior panels		Interior panels
	Without edge beams	With edge beams§		Without edge beams	With edge beams§	
72,500	$l_n/28.33$	$l_n/31.33$	$l_n/31.33$	$l_n/31.33$	$l_n/34.33$	$l_n/34.33$

*For two-way construction, l_n is the length of clear span in the long direction, measured face-to-face of supports in slabs without beams and face-to-face of beams or other supports in other cases.
‡Drop panels as defined in Section 13.2.5 of ACI Code 318-11
§Slabs with beams between columns along exterior edges. The value of α_f for the edge beam shall not be less than 0.8.

4.4.3. Two-way slabs with beams

Reference: Section 9.5.3.3

Minimum thickness for two-way slabs with beams is:

For $\alpha_{fm} \leq 0.2$, h will remain same as defined in Section 4.4.2 of this manual.

For $2.0 > \alpha_{fm} > 0.2$, h shall not be less than

$$h = \frac{l_n \left(0.8 + \frac{f_y}{200,000} \right)}{36 + 5\beta (\alpha_{fm} - 0.2)} \dots \dots \dots (9 - 12)$$

But not less than 5 inches.

For $\alpha_{fm} > 0.2$, h shall not be less than

$$h = \frac{l_n \left(0.8 + \frac{f_y}{200,000} \right)}{36 + 9\beta} \dots \dots \dots (9 - 13)$$

But not less than 3.5 inches.

4.5. Minimum reinforcement for flexural members

Reference: Section 10.5

Minimum reinforcement requirement for flexural members as per ACI Code is:

$$A_{s, \min} = \frac{3\sqrt{f'_c}}{f_y} b_w d \dots\dots\dots (10 - 3)$$

And not less than $200 b_w d / f_y$.

The application of the above equation is valid except as outlined in sec. 10.5.2, 10.5.3 and 10.5.4 of the ACI Code.

For typical values of f'_c , the above equations can be simplified as shown in Table 4.5.

TABLE 4.5- MINIMUM REINFORCEMENT FOR FLEXURAL MEMBERS

$A_{s, \min}$ Larger of A or B							
f'_c (PSI)	4000	4500	5000	5500	6000	6500	7000
A (Eq. 10-3, ACI 318-11)	$0.00262bd$	$0.00278bd$	$0.00293bd$	$0.00307bd$	$0.00321bd$	$0.00336bd$	$0.00346bd$
B ($200 b_w d / f_y$)	$0.00276bd$						

4.6. Distribution of flexural reinforcement in beams and one-way slabs

Reference: Section 10.6.4

The spacing of reinforcement closest to the tension face, s , shall not exceed that given by

$$s = 15 \left(\frac{40,000}{f_s} \right) - 2.5 c_c \dots\dots\dots (10 - 4)$$

But not greater than $12(40,000/f_s)$, where c_c is the least distance from surface of reinforcement or pre-stressing steel to the tension face. If there is only one

bar or wire nearest to the extreme tension face, s used in above equation is the width of the extreme tension face. Calculated stress f_s in reinforcement closest to the tension face at service load shall be computed based on the unfactored moment. It is permitted to take f_s as $2/3f_y$ by ACI Code. For typical values of clear cover c_c , and taking f_s equals to $2/3$ of 72,500 PSI i.e **48,333.333 PSI**, the above equation can be simplified as given in Table 4.6.

TABLE 4.6- MAXIMUM SPACING OF FLEXURAL REINFORCEMENT IN BEAMS AND ONE-WAY SLABS

s	c_c (inch)	0.75	1	1.5	2	2.5	3
		By Eq. 10-4 (ACI 318-11)	10.54	9.91	8.66	7.41	6.16

4.7. Development of deformed bars and deformed wire in tension

Reference: Section 12.2

Development length for deformed bars and wires in tension, l_d in ACI Code is given by the following equations, but not less than 12”.

Spacing and Cover	No. 6 and smaller bars and deformed wires	No. 7 and larger bars
Clear spacing of bars or wires being developed or spliced not less than d_b , clear cover not less than d_b , and stirrups or ties throughout l_d not less than the Code minimum, Or Clear spacing of bars and wires being developed or spliced not less than $2d_b$ and clear cover not less than d_b	$\left(\frac{f_y \Psi_t \Psi_e}{25 \lambda \sqrt{f'_c}}\right) d_b$	$\left(\frac{f_y \Psi_t \Psi_e}{20 \lambda \sqrt{f'_c}}\right) d_b$
Other Cases	$\left(\frac{3 f_y \Psi_t \Psi_e}{50 \lambda \sqrt{f'_c}}\right) d_b$	$\left(\frac{3 f_y \Psi_t \Psi_e}{40 \lambda \sqrt{f'_c}}\right) d_b$

Using the above equations, for typical conditions where, $f_y = 72.5$ KSI, $\lambda = 1.0$ for normal weight concrete, $\Psi_e = 1.0$ for uncoated reinforcement, $\Psi_t = 1.0$ for bottom bars, and clear spacing of bars or wires being developed or spliced not less than d_b , clear cover not less than d_b , and stirrups or ties throughout l_d not less than the Code minimum or clear spacing of bars or wires being developed or spliced not less than $2d_b$ and clear cover not less than d_b , the development lengths for bars in

combination with different f'_c values may be generally estimated as given in Table 4.7(a).

TABLE 4.7(a) - DEVELOPMENT LENGTHS FOR DEFORMED BARS IN TENSION, HAVING YIELD STRENGTH OF 72.5 KSI

(Note: values are rounded off to whole number)

l_d	f'_c (PSI)	4000	4500	5000	5500	6000	6500	7000
	For No.6 and smaller bars and wires:	$46d_b$	$43d_b$	$41d_b$	$39d_b$	$37d_b$	$36d_b$	$35d_b$
	For No.7 and larger bars:	$57d_b$	$54d_b$	$51d_b$	$49d_b$	$47d_b$	$45d_b$	$43d_b$

For other cases of spacing and cover, development length is given as shown in Table 4.7(b).

TABLE 4.7(b) - DEVELOPMENT LENGTHS FOR DEFORMED BARS IN TENSION, HAVING YIELD STRENGTH OF 72.5 KSI

l_d	f'_c (PSI)	4000	4500	5000	5500	6000	6500	7000
	For No.6 and smaller bars and wires:	$69d_b$	$65d_b$	$62d_b$	$59d_b$	$56d_b$	$54d_b$	$52d_b$
	For No.7 and larger bars:	$86d_b$	$81d_b$	$77d_b$	$73d_b$	$70d_b$	$67d_b$	$65d_b$

For top bars, $\Psi_t = 1.3$ (refer Section 12.2.4(a) of ACI Code 318), and clear spacing of bars or wires being developed or spliced not less than d_b , clear cover not less than d_b , and stirrups or ties throughout l_d not less than the Code minimum or clear spacing of bars or wires being developed or spliced not less than $2d_b$ and clear cover not less than d_b , the development lengths for bars in combination with different f'_c values may be generally estimated as given in Table 4.7(c).

TABLE 4.7(c) - DEVELOPMENT LENGTHS FOR DEFORMED BARS IN TENSION, HAVING YIELD STRENGTH OF 72.5 KSI

l_d	f'_c (PSI)	4000	4500	5000	5500	6000	6500	7000
	For No.6 and smaller bars and wires:	$60d_b$	$56d_b$	$53d_b$	$51d_b$	$49d_b$	$47d_b$	$45d_b$
	For No.7 and larger bars:	$75d_b$	$70d_b$	$67d_b$	$64d_b$	$61d_b$	$58d_b$	$56d_b$

For top bars and for other cases of spacing and cover, development lengths are given as listed in Table 4.7(d).

TABLE 4.7(d) - DEVELOPMENT LENGTHS FOR DEFORMED BARS IN TENSION, HAVING YIELD STRENGTH OF 72.5KSI

l_d	f'_c (PSI)	4000	4500	5000	5500	6000	6500	7000
	For No.6 and smaller bars and wires:	$89d_b$	$84d_b$	$80d_b$	$76d_b$	$73d_b$	$70d_b$	$68d_b$
	For No.7 and larger bars:	$112d_b$	$105d_b$	$100d_b$	$95d_b$	$91d_b$	$88d_b$	$84d_b$

Note: Alternate provision given in Section 12.2.3 of ACI Code results in lesser development length than calculated by Section 12.2.2 of ACI Code; hence it is not shown here.

4.8. Development of deformed bars and deformed wire in compression

Reference: Section 12.3

As per ACI Code, for deformed bars and deformed wire, l_{dc} shall be taken as the larger of:

$$\left(\frac{0.02 f_y}{\lambda \sqrt{f'_c}}\right) d_b \text{ and } (0.0003 f_y) d_b$$

with λ as given in Section 12.2.4(d) of ACI Code 318-11. Constant 0.0003 carries the unit of in²/lb.

For typical conditions where, $f_y = 72.5$ KSI, $\lambda = 1.0$ for normal weight concrete, development length of deformed bars in compression (l_{dc}) having yield strength 72.5 KSI is calculated as shown in Table 4.8, but should not be less than 8 inches.

TABLE 4.8- DEVELOPMENT LENGTHS FOR DEFORMED BARS IN COMPRESSION, HAVING YIELD STRENGTH OF 72.5KSI

l_{dc} (Larger of A or B)	f'_c (PSI)	4000	4500	5000	5500	6000	6500	7000
	A	$23 d_b$	$22 d_b$	$21 d_b$	$20 d_b$	$19 d_b$	$18 d_b$	$17 d_b$
	B	$22 d_b$						

Note: Reduction in l_{dc} is allowed as per Section 12.3.3 of ACI Code 318-11.

4.9. Development of standard hooks in tension

Reference: Section 12.5

For deformed bars, l_{dh} shall be $(0.02\Psi_e f_y / \lambda\sqrt{f'_c}) d_b$ with Ψ_e taken as 1.2 for epoxy-coated reinforcement, and λ taken as 0.75 for lightweight concrete. For other cases, Ψ_e and λ shall be taken as 1.0, but not less than the larger of $8d_b$ and 6 inches.

For typical case with normal weight, concrete and uncoated reinforcement, values of l_{dh} for different values of f'_c , are given in Table 4.9.

TABLE 4.9- DEVELOPMENT LENGTH OF STANDAD HOOKS IN TENSION FOR DEFORMED BARS HAVING 72.5 KSI YIELD STRENGTH

l_{dh}	f'_c (PSI)	4000	4500	5000	5500	6000	6500	7000
			$23 d_b$	$22 d_b$	$21 d_b$	$20 d_b$	$19 d_b$	$18 d_b$

Note: l_{dh} is permitted to be reduced as per Section 12.5.3 of ACI Code 318-11.

4.10. Splices of deformed bars and deformed wire in tension

Reference: Section 12.15

Minimum length of lap for tension lap splices shall be as required for Class A or B splice, but not less than 12 in., where:

Class A splice 1.0 l_d
Class B splice 1.3 l_d

where l_d is calculated in accordance with Section 4.7 as noted earlier in this Chapter, to develop f_y , but without the 12 in. minimum as required in Section 12.2.1 of the Code and without the modification factor of excess reinforcement given in Section 12.2.5 of Code.

4.11. Splices of deformed bars in compression

Reference: Section 12.16

Compression lap splice length shall be $(0.0009 f_y - 24)d_b$ for f_y greater than 60,000 PSI, but not less than 12 in.

Therefore, for $f_y = 72.5 \text{ KSI}$, splice length for deformed bars in compression = $41.25d_b$.

4.12. Wall Design

Reference: Section 14.3

Minimum ratio of vertical reinforcement area to gross concrete area ρ_l shall be:

- (a) *0.0012 for deformed bars not larger than No. 5 with f_y not less than 60,000 PSI; or*
- (b) *0.0015 for other deformed bars.*

Using above ratios, for typical values of wall thickness, minimum area (sq.in/ft.) of G-500 steel required for vertical reinforcement is given in Table 4.12(a).

TABLE 4.12(a) - MINIMUM RATIO OF VERTICAL REINFORCEMENT AREA TO GROSS CONCRETE AREA FOR BARS HAVING YIELD STRENGTH OF 72.5 KSI

	Wall Thickness (in.)								
	10	12	14	15	16	18	20	21	24
For Bar dia \leq #5, $A_{s,min}$ (sq. in/ft.)	0.144	0.172	0.201	0.216	0.230	0.259	0.288	0.302	0.345
For Bar dia $>$ #5, $A_{s,min}$ (sq. in/ft.)	0.18	0.216	0.252	0.27	0.288	0.324	0.36	0.378	0.432

Minimum ratio of horizontal reinforcement area to gross concrete area ρ_t shall be:

- (a) *0.0020 for deformed bars not larger than No. 5 with f_y not less than 60,000 PSI; or*
- (b) *0.0025 for other deformed bars.*

Using above ratios, for typical values of wall thickness, minimum area (sq.in/ft.) of G-500 steel required for horizontal reinforcement is given in Table 4.12(b).

TABLE 4.12(b) - MINIMUM RATIO OF HORIZONTAL REINFORCEMENT AREA TO GROSS CONCRETE AREA FOR BARS HAVING YIELD STRENGTH OF 72.5 KSI

	Wall Thickness (in.)								
	10	12	14	15	16	18	20	21	24
For Bar dia \leq #5, $A_{s,min}$ (sq. in/ft.)	0.24	0.288	0.336	0.36	0.384	0.432	0.48	0.504	0.576
For Bar dia $>$ #5, $A_{s,min}$ (sq. in/ft.)	0.3	0.36	0.42	0.45	0.48	0.54	0.6	0.63	0.72

Chapter 5

Limitations on use of High Strength Steel Based on ACI Code

ACI Code 318-11 restricts use of yield strength higher than 60 KSI for some design applications. Some considerations that need to be taken into account while designing with high strength steel are listed in this Chapter. Guidelines as listed in Section 4.1 of this manual are same for this Chapter also.

5.1. Material Specification

Reference: Section 3.5.3.2

G500 (72.5KSI yield strength) deformed reinforcing bars are not covered by any American standard.

5.2. Redistribution of Moments

Reference: Section 8.4

Redistribution of moments in continuous members is covered in clause 8.4.1 and 8.4.2 of ACI Code 318-11. It is interpreted that for 60 and higher yield strengths, same clauses apply for moment redistribution.

5.3. Limits for spiral reinforcement of compression members

Reference: Section 10.9.3

Volumetric spiral reinforcement ratio, ρ_s shall be not less than the value given by;

$$\rho_s = 0.45 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}} \dots \dots \dots (10 - 5)$$

Where the value of f_{yt} used in Eq. (10-5) shall not exceed 100,000 PSI.

Two of the functions of transverse reinforcement in a reinforced concrete member are to confine the concrete and to act as shear reinforcement. For confinement purposes, the upper limit on f_{yt} is 100,000 PSI.

5.4. Shear and Torsion

Reference: Section 11.4.2 & Section 11.5.3.4

“The values of f_y and f_{yt} used in design of shear reinforcement shall not exceed 60,000 PSI, except the value shall not exceed 80,000 PSI for welded deformed wire reinforcement.”

“The values of f_y and f_{yt} used for design of torsional reinforcement shall not exceed 60,000 PSI.”

The upper limit on the yield strength of transverse reinforcement is imposed to limit the width of possible shear cracks to acceptable levels. This restriction does not mean that higher-strength bars may not be used as transverse reinforcement. It simply means that a higher yield strength may not be used for the calculation of shear strength [7].

5.5. Development of headed and mechanically anchored deformed bars in tension

Reference: Section 12.6

The development of headed deformed bars and the development and anchorage of reinforcement through use of mechanical devices within concrete is not permitted for reinforcing bars having f_y greater than 60 KSI.

5.6. Pre-stressed Concrete

Reference: Section 18.9.3.2

“In positive moment areas where computed tensile stress in concrete at service load exceeds $2\sqrt{f'_c}$, minimum area of bonded reinforcement shall be computed by:

$$A_s = \frac{N_c}{0.5f_y} \dots \dots \dots (18 - 5)$$

Where, the value of f_y used in Eq. (18-7) shall not exceed 60,000 PSI.”

5.7. Shells and Folded Plate Members

Reference: Section 19.3.2

“Specified yield strength of non-prestressed reinforcement f_y shall not exceed 60,000 PSI.”

5.8. Ordinary and Intermediate Moment Resisting frames

Reference: Section 21.1

Earthquake resistant structures designed, as Ordinary moment resisting frames and Intermediate moment resisting frames do not have any special requirements as far as material property for reinforcing bars are concerned.

5.9. Special moment Resisting frames and Walls

Reference: Section 21.1.5 & 21.1.1.8

In ACI Code, only Grade 40 or 60 reinforcement conforming to ASTM A615, or G-60 reinforcement conforming to ASTM A706 is allowed in special moment frames and structural walls.

ACI Code places a ceiling of 60-KSI yield strength because of insufficient data to confirm applicability of existing Code provisions for structures using the higher grade. Section 21.1.1.8 permits alternative material if results of tests and analytical studies are presented.

References

1. Munaz Ahmed Noor, 2010, "Designing with Grade 500 Steel", the university Press Limited, Dhaka, Bangladesh.
2. British Standard, 2005, "Steel for the reinforcement of concrete, weldable reinforcing steel, bar, coil and decoiled product Specifications," BS 4449.
3. R. Park & T. Pauly, 1974, "Reinforced Concrete Structures", Wiley-Interscience Publication.
4. American Concrete institute, 2011, "Building Code Requirements for Structural concrete and commentary-ACI 318-11".
5. ACI Committee 224, Reapproved 2008, "Control of Cracking in Concrete Structures".
6. M.Nadim Hassoun, 1985, "Design Of Reinforced Concrete Structures", PWS (Prindlr, Weber & Schmidt) Publication.
7. S. K. Ghosh, Ph.D., December 2011, "Restrictions on the strength of reinforcement in ACI 318-08",

APPENDIX-A ACTS TEST REPORTS

Build on our credentials

TEST REPORT

Tested at : ACTS Central Laboratory, Qatar
Street 13, Gate 172, Industrial Area
P.O.Box: 22159 Doha, Qatar
Tel: 974-44601257; Fax: 974-44601254

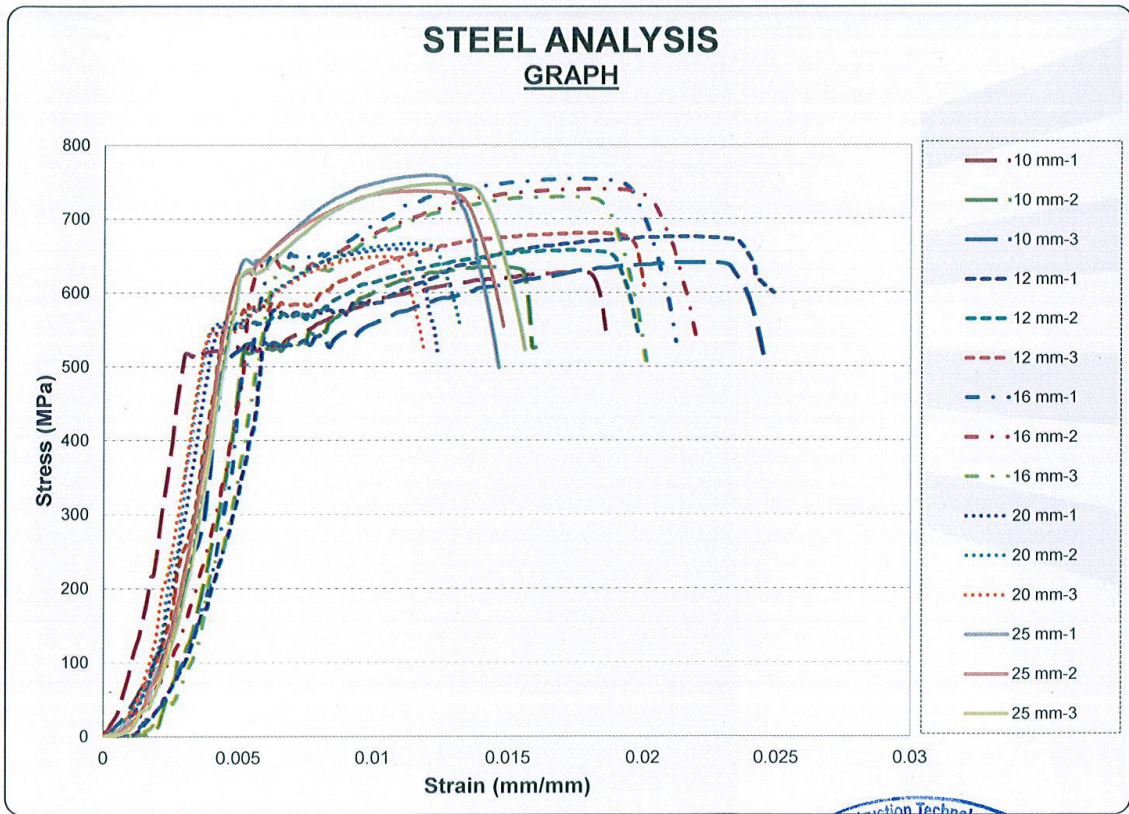
Customer : Sadaf Fatima Consulting Engineer
86-B, 2nd Floor, Muhammad Ali
Society Karachi 75350 Pakistan
Tel. No. +92-21-34299828
Mobile No.+974-55615100

STEEL TEST FOR REINFORCEMENT CONCRETE

(ASTM A370/A615)

Contractor : Sadaf Fatima Consulting Engineer
Consultant : Not Specified
Project : Evaluation of Materials
ACTS Ref. : DEC14080904-0931

Work Order No. : 1428690
Sample received on : 08/12/2014
Sample tested on : 14/12/2014
Sampled by : Customer



Reviewed By : Hussein Ibrahim
Head of Department




LEGAL NOTICE

ACTS shall bear no responsibility for the authenticity of the sample, which was the basis of this report, unless the sampling is conducted by ACTS employees. This will be clearly indicated in the relative test report. The information given in ACTS reports is for the use of the client. It is not to be abstracted or published by any means or in any form, in whole or in part. Prior written consent from ACTS is required in the case of necessity for use of the report. It is strictly prohibited to use this report for advertising purposes. ACTS shall not assume any responsibility for any interpretations of the report except when the interpretation is given in writing by ACTS. Neither ACTS nor its employees shall bear any responsibility for the damage, loss, omission, failure, or injury; arising directly or indirectly in connection with ACTS performance testing, inspection, calibration of materials, instruments, or other articles.

IMS Certified

ISO 9001
ISO 14001
OHSAS 18001

rtified

Build on our credentials

TEST REPORT

Reviewed at : ACTS Central Laboratory, Qatar
Street 13, Gate 172, Industrial Area
P.O.Box: 22159 Doha, Qatar
Tel: 974-44601257; Fax: 974-44601254

Customer : Sadaf Fatima Consulting Engineer
82-B 2nd Floor Mohammad Ali Society
Karachi
P O Box : 75350, Pakistan, PAKISTAN
Tel : 0092-300-3572046 Fax :

CHEMICAL ANALYSIS OF STEEL

Contractor : -----
Consultant : -----
Project : **Evaluation of Materials**
Sample Type : Steel
Customer Ref. : 10mm
ACTS Ref. : DEC14080910

Work Order No. : 1428690
Sample received on : 08/12/2014
Sample Tested on : 29/01/2015
Sampled by : Customer

Sampling details Date of Sampling : 02/12/2014

Element	Symbol	Results*	BS 4449 Limits
Carbon, %	C	0.227	Max. 0.24
Silicon, %	Si	0.205	-----
Manganese, %	Mn	0.701	-----
Chromium, %	Cr	0.103	-----
Nickel, %	Ni	0.053	-----
Copper, %	Cu	0.126	Max. 0.85
Molybdenum, %	Mo	0.016	-----
Phosphorous, %	P	0.036	Max. 0.055
Sulfur, %	S	0.031	Max. 0.055
Vanadium, %	V	0.002	-----

Remark : The analysis was done based on X-ray technique.

: *Test subcontracted to a competent laboratory and approved by ACTS management.

REVIEWED BY: *Alhynn Tesorero*
Head of Department



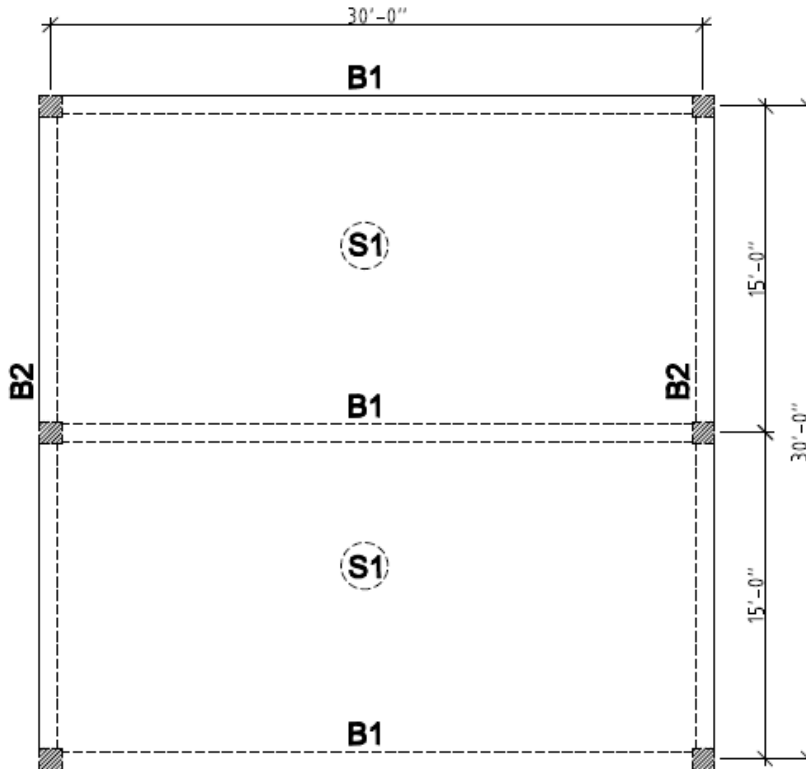
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APPENDIX-B
SOLVED EXAMPLE FOR
G500 REINFORCEMENT



Example: As shown in Figure above, a simply supported beam, B1 and slab, S1 are designed using G500 reinforcement. Beam width is 10 inches. Compressive strength of concrete is taken as 4 KSI. Yield strength of reinforcing bars is 72.5 KSI (500 MPa)

Loads:

- Self-Weight
- Super imposed dead load = 36 p.s.f
- Live load = 50 p.s.f
- $F'c = 4$ K.S.I
- $Fy = 72.5$ K.S.I
- Column size = 12"x12"
- Beam width = 10"

Solution:

By table 4.4.1m of this manual, minimum thickness of slab S1 = Span/21.3

$$\Rightarrow \frac{15 \times 12}{21.3} = 8.45 \sim 8.5" \text{ thick.}$$

By table 4.4.1m of this manual, Minimum depth of B1 = Span/14.2 = 25.35" ~ 30"

Slab Design:

$$\text{Self weight} = \frac{150 \times 8.5}{12} \sim 107 \text{ p.s.f}$$

Super-imposed Dead load = 36 psf

Live load = 50 psf

$$\Rightarrow Wu = 1.2 (107+36) + 1.6(50) = 251.6 \sim 252 \text{ psf}$$

- ⇒ Max. +ve moment = $\frac{Wu \times l^2}{14} = \frac{0.252 \times 15^2}{14} = 4.085 \text{ K-ft/ft}$ or 48.6 K-in/ft
- ⇒ Max. -ve moment = $\frac{Wu \times l^2}{9} = \frac{0.252 \times 15^2}{9} = 6.3 \text{ K-ft/ft}$ or 75.6 K-in/ft
- ⇒ $Ru = \phi \rho f_y \left(1 - \frac{\rho f_y}{1.7 f'_c}\right)$
- ⇒ $Ru = 0.9 \times \rho \times 72.5 \left(1 - \frac{\rho \times 72.5}{1.7 \times 4}\right)$
- ⇒ $Ru = 65.25\rho - 695.68\rho^2$
- ⇒ $As Mu = Ru bd^2 = \phi Mn$
- ⇒ For +ve moment,
- ⇒ $48.6 = (65.25\rho - 695.68\rho^2) \times (12 \times 7.75^2)$
- ⇒ $695.68\rho^2 - 65.25\rho + 0.0674 = 0$
- ⇒ Solving quadratic equation for ρ ,
- ⇒ $\rho = 0.0927$, or $\rho = 1.0445 \times 10^{-3}$
- ⇒ $As = \rho bd$
- ⇒ $As = 1.0445 \times 10^{-3} \times 12 \times 7.75 = 0.097 \text{ in}^2/\text{ft}$
- ⇒ $As(\text{min}) = 0.00149 \times 12 \times 8.5 = 0.15198 \text{ in}^2/\text{ft}$
- ⇒ Provide #3 @ 9" c/c Bottom Reinforcement.

- ⇒ For -ve moment,
- ⇒ $48.6 = (65.25\rho - 695.68\rho^2) \times (12 \times 7.75^2)$
- ⇒ $695.68\rho^2 - 65.25\rho + 0.105 = 0$
- ⇒ Solving quadratic equation for ρ ,
- ⇒ $\rho = 0.092$, or $\rho = 1.638 \times 10^{-3}$
- ⇒ $As = \rho bd$
- ⇒ $As = 1.638 \times 10^{-3} \times 12 \times 7.75 = 0.15233 \text{ in}^2/\text{ft}$
- ⇒ Provide #3 @ 9" c/c Top Reinforcement.
- ⇒ Distribution #3 @ 9" c/c

Beam Design for B1:

$$\text{Self-weight} = \frac{10 \times 30 \times 0.15}{144} = 0.3125 \text{ k/ft}$$

$$\text{Super-imposed dead load} = \frac{(107+36) \times 15}{1000} = 2.145 \text{ k/ft}$$

$$\text{Live load} = \frac{50 \times 15}{1000} = 0.75 \text{ k/ft}$$

- ⇒ $Wu = 1.2(2.145 + 0.3125) + 1.6(0.75) = 4.15 \text{ k/ft}$.
- ⇒ Max. +ve moment @ mid-span = $\frac{Wu \times l^2}{8} = \frac{4.15 \times 30^2}{8} = 466.875 \text{ K-ft}$.
- ⇒ For +ve moment,
- ⇒ $Mu = \phi As f_y \left(d - \frac{As f_y}{1.7 f'_c b}\right) = \phi Mn$
- ⇒ $466.875 \times 12 = 0.9 \times As \times 72.5 \left(26.5 - \frac{As \times 72.5}{1.7 \times 4 \times 10^3}\right)$
- ⇒ $5602.5 = 1729.125As - 69.56As^2$
- ⇒ $69.56As^2 - 1729.125As + 5602.5 = 0$
- ⇒ Solving quadratic equation for As ,
- ⇒ $As = 21.03 \text{ in}^2$, or 3.83 in^2
- ⇒ Check,

$$\Rightarrow \phi Mn = 0.9 \times 3.83 \times 72.5 \left(26.5 - \frac{3.83 \times 72.5}{1.7 \times 4 \times 10} \right) = 5602.5 \text{ k-in (OK)}$$

Check for A_s (min), greater of,

$$A_s(\text{min.}) = 3\sqrt{f'_c} \frac{bd}{f_y} = 3\sqrt{4000} \frac{10 \times 26.5}{72500} = 0.6935 \text{ in}^2$$

$$\text{Or, } \frac{200 \times b \times d}{f_y} = \frac{200 \times 10 \times 26.5}{72500} = 0.731 \text{ in}^2$$

$$\Rightarrow A_s(\text{min.}) = 0.731 \text{ in}^2$$

Check for A_s (max),

$$A_s(\text{max.}) = 0.75 \left[0.85\beta_1 \frac{f'_c}{f_y} \left(\frac{87}{87 + f_y} \right) bd \right] = 4.32 \text{ in}^2$$

$$\Rightarrow A_s = 3.83 \text{ in}^2$$

\Rightarrow Provide 4#6 bottom bars throughout + 4#6 extra bottom

\Rightarrow Using 2#6 @ top reinforcement,

\Rightarrow Provide 2 #6 top bars throughout.

APPENDIX-C

BAR RECKONER

AMRELI STEELS LIMITED

BAR RECKONER DEFORMED BARS (IMPERIAL SIZES) ASTM STANDARD

Bar Designation No.	Bar Size In mm	Bar Area In mm ²	Weight kg / ft	Weight kg / Mtr	Weight of 40' Bar (Kg)	Weight of 12 Mtr Bar (Kg)	Rft / Ton (Approx)	Mtr / Ton (Approx)
3	9.5	71	0.171	0.56	6.8	6.7	5859	1786
4	12.7	129	0.303	0.994	12.1	11.9	3301	1006
5	15.9	199	0.473	1.552	18.9	18.6	2114	644
6	19.1	284	0.681	2.235	27.2	26.8	1468	447
7	22.2	387	0.927	3.042	37.1	36.5	1079	329
8	25.4	510	1.211	3.973	48.4	47.7	826	252
9	28.7	645	1.542	5.06	61.7	60.7	648	198
10	32.3	819	1.952	6.404	78.1	76.8	512	156
11	35.8	1006	2.410	7.907	96.4	94.9	415	126

BAR RECKONER DEFORMED BARS (METRIC SIZES) ASTM STANDARD

Bar Size In mm	Bar Area In mm ²	Weight kg / ft	Weight kg / Mtr	Weight of 40' Bar (Kg)	Weight of 12 Mtr Bar (Kg)	Rft / Ton (Approx)	Mtr / Ton (Approx)
10	79	0.188	0.617	7.5	7.4	5317	1621
12	113	0.271	0.888	10.8	10.7	3695	1126
16	201	0.481	1.578	19.2	18.9	2079	634
20	314	0.752	2.466	30.1	29.6	1330	406
25	491	1.174	3.853	47.0	46.2	852	260
28	615	1.473	4.834	58.9	58.0	679	207
32	804	1.924	6.313	77.0	75.8	520	158
36	1018	2.435	7.990	97.4	95.9	411	125
40	1256	3.007	9.865	120.3	118.4	333	101

BAR RECKONER XTREME BARS G-500 W BRITISH STANDARD

Bar Size In mm	Bar Area In mm ²	Weight kg / ft	Weight kg / Mtr	Weight of 40' Bar (Kg)	Weight of 12 Mtr Bar (Kg)	Rft / Ton (Approx)	Mtr / Ton (Approx)
9.5	71	0.171	0.56	6.8	6.7	5859	1786
12	113	0.271	0.888	10.8	10.7	3695	1126
16	201	0.481	1.578	19.2	18.9	2079	634
19	284	0.678	2.226	27.1	26.7	1474	449
22	380	0.910	2.984	36.4	35.8	1099	335
25	491	1.174	3.853	47.0	46.2	852	260