공학석사학위논문

Evaluation of Applicability for Code Provisions Related to Crack and Deflection on SD700

균열 및 처짐의 규정에 대한 SD700 철근 적용성 평가

2021 년 2 월

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이 논문을 공학석사 학위논문으로 제출함
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ABSTRACT

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In the civil engineering society, traditional construction materials, such as timber, steel, asphalt, and portland cement concrete are often used in many construction projects. Significant research on these materials has led to an understanding of these materials and improved their strength and durability performance. The traditional materials used today are far superior to those of the past, and new materials are being specially developed to satisfy the needs of civil engineering applications.

Most concrete used for construction is a combination of concrete and reinforcement, with steel being the most common material used as reinforcement. Reinforced concrete is typically used for large-scale infrastructure. For such infrastructure projects, the reinforcement must be of the right kind, of the right amount. When normal strength rebar is used for these structures, a large amount of reinforcement is needed to meet requirements for strength and serviceability. However, excessive reinforcement may reduce

concrete quality. One suitable way to avoid this issue is through the application of high strength rebar, which provides various benefits.

On the other hand, other issues may arise when using high strength rebar for reinforced concrete structures. Reduced reinforcement congestion when using high strength rebar increases rebar spacing, which can result in corresponding increases in crack width and deflection. Thus, serviceability of reinforced concrete is an important consideration when using high strength rebar. In light of this, the maximum yield strength of rebar is limited in concrete design codes. In Korean concrete design codes, SD600 rebar has already been allowed. Research on higher strength SD700 rebar has also been conducted, but it has not yet been included in the Korean concrete design codes due to need for additional research on serviceability.

This study seeks to address this important research need. In this study, flexural tests were conducted to evaluate the applicability for code provisions related to crack and deflection on SD700. RC beam and one way slabs reinforced with SD700 were tested. Allowable crack width and minimum height of members presented in KDS are checked for evaluation of serviceability provisions.

Keywords: high strength rebar, yield strength of rebar, allowable crack width, minimum height of member, effective stiffness

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NOTATIONS

Symbol		Definition and description
A	=	Effective tension area of concrete surrounding the flexural tension reinforcement and extending from the extreme tension fibre to the centroid of the flexural tension reinforcement and an equal distance past that centroid, divided by the number of bars or wires
$A_{c,eff}$	=	Effective area of concrete in tension surrounding the reinforcement or prestressing tendons of depth
A_{ct}	=	Area of concrete within tensile zone
A_s	=	Area of tensile reinforcement
$A_s^{'}$	=	Area of compression reinforcement
$A_{s,\min}$	=	Minimum cross-sectional area of reinforcement
$A_p^{'}$	=	Area of pre or post-tensioned tendons within $A_{c,eff}$
B	=	Width of member
b	=	Width of compression face of member
b_w	=	Web width or diameter of circular section
c_c	=	Clear cover of reinforcement
c	=	Depth of the neutral axis
d	=	Effective depth
d	=	Distance from extreme compression fiber to centroid of longitudinal tension reinforcement

d'	=	Distance from extreme compression fiber to centroid of longitudinal compression reinforcement
d_c	=	Distance from extreme tension fiber to center of the longitudinal bar or wire located closest to it
E	=	Young's modulus
EI_e	=	Flexural stiffness
E_{cm}	=	Secant modulus of elasticity of concrete
E_s	=	Design value of modulus of elasticity of reinforcing steel
$f_c^{'}, f_{ck}$	=	Compressive strength of concrete
F_{cr}	=	Absolute value of the tensile force within the flange immediately prior to cracking due to the cracking moment calculated with $f_{ct,eff}$
$f_{ct}, f_{ct,eff}$	=	Mean value of the tensile strength of the concrete effective at the time when the cracks may first be expected to occur
f_y	=	Specified yield strength for non-prestressed reinforcement
f_{pe}	=	Compressive stress in concrete due only to effective prestress forces, after allowance for all prestress losses, at extreme fiber of section if tensile stress is caused by externally applied loads
f_r	=	Modulus of rupture of concrete
f_s	=	Tensile stress in reinforcement at service load
f_{ss}	=	Calculated tensile stress in non-prestressed reinforcement at the service limit state
f_{so}	=	Tensile reinforcement stress calculated based on the crack section

f_{sr}	=	Tensile stress of the reinforcing bar calculated at the crack face immediately after the first crack occurs
H	=	Height of member
$h_{\min}$	=	Minimum height of member
I_{cr}	=	Moment of inertia of cracked section transformed to concrete
I_e	=	Effective moment of inertia for calculation of deflection
I_g	=	Moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement
K	=	Factor to take into account the different structural systems
k	=	Coefficient which allows for the effect of non-uniform self-equilibrating stresses, which lead to a reduction of restraint forces
kd	=	Neutral axis
k_c	=	Coefficient which takes account of the stress distribution within the section immediately prior to cracking and of the change of the lever arm
k_{cr}	=	Exposed conditions coefficient of rebar
k_t	=	Factor dependent on the duration of the load
k_1	=	Coefficient considering the effects of axial forces on the stress distribution
L	=	Length of member
l	=	Span length of beam or one way slab
$l_{r,\max}$	=	Maximum crack spacing
M_a	=	Maximum moment in member due to service loads at stage deflection is calculated

M_{cr}	=	Cracking moment
M_n	=	Nominal flexural strength at section
M_s	=	Flexural moment at section under service load
n	=	Number of reinforcing bars
N_{crack}	=	Deformation in completely cracked state
N_{cr}	=	Cracking force
N_{Ed}	=	Axial force at the serviceability limit state acting on the part of the cross-section under consideration (compressive force positive)
S	=	Slab span length
s	=	The spacing of non-prestressed reinforcement in the layer closest to the tension face
y_t	=	Distance from centroidal axis of gross section, neglecting reinforcement, to tension face
z	=	Quantity limiting distribution of flexural reinforcement
Δ_e	=	Average effective strain over the span before absence
Δ_e	=	Deformation parameter considered which may be, for example, a strain, a curvature, or a rotation
Δ_u	=	Displacement at maximum load
$\Delta_{uncrack}$	=	Deformation in non-cracked state
Δ_y	=	Displacement at yielding of tensile rebar
$\Delta\sigma_p$	=	Stress variation in prestressing tendons

α_e	=	Ratio E_s / E_{cm}
β	=	Factor reflecting the repeated duration of the load on the average strain
β_s	=	Ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face
γ_c	=	Density of concrete
γ_e	=	Exposure factor
ε_c	=	Strain in concrete
ε_{cm}	=	Mean strain in the concrete between cracks
ε_s	=	Strain in reinforcement
ε_{sm}	=	Mean strain in the reinforcement under the relevant combination of loads
ε_t	=	Net tensile strain
ζ	=	Distribution coefficient
λ	=	Factor to account for low density concrete
λ_Δ	=	Multiplier used for additional deflection due to long term effects
μ_Δ	=	Ductility ratio
ξ	=	Time dependent factor for sustained load
ξ_1	=	Adjusted ratio of bond strength taking into account the different diameters of prestressing and reinforcing steel

ρ	=	Ratio of non-prestressed tension reinforcement, equal to A_s / bd
ρ'	=	Reinforcement ratio for compression reinforcement, equal to A_s' / bd
ρ_0	=	Reference reinforcement ratio
σ_c	=	Mean stress of the concrete acting on the part of the section under consideration
σ_s	=	Stress in the tension reinforcement calculated on the basis of a cracked section
σ_{sr}	=	Stress in the tension reinforcement calculated on the basis of a cracked section
ϕ	=	Bar diameter
ϕ_{eq}	=	Equivalent diameter of bar
ϕ_p	=	Equivalent diameter of tendon
ϕ_s	=	Curvature under service load
ϕ_s	=	Largest bar diameter of reinforcing steel
ω_k	=	Crack width

1. Introduction

1.1. Research Background

Construction of large-scale infrastructure projects is increasing in the modern society. This includes large-scale infrastructure projects such as dams, canals, airports, long span bridges, nuclear power plants, skyscrapers, etc. These projects require the use of high strength material which decreases material thickness while maintaining or even increasing performance. High strength material must satisfy demanding technical requirements as well as the need for good handling during construction even as structure height and span continue to get taller and longer. Therefore, high strength material is necessary for analysis and design of large-scale infrastructures.

Typically, reinforced concrete structures have been used for existing large-scale infrastructures. When using normal strength rebar for large-scale infrastructures, a large quantity of reinforcement is needed. However, excessive reinforcement diminishes the quality of concrete. Opting for high strength rebar is one method for avoiding excessive reinforcement by decreasing the amount of rebar required (Lee et al 2010).

Normal strength rebar is commonly used in construction. The mechanical properties of normal and high strength rebar differ significantly. High strength rebar is not more ductile compared to normal strength rebar. Instead, the brittle behavior of rebar increases as yield strength increases.

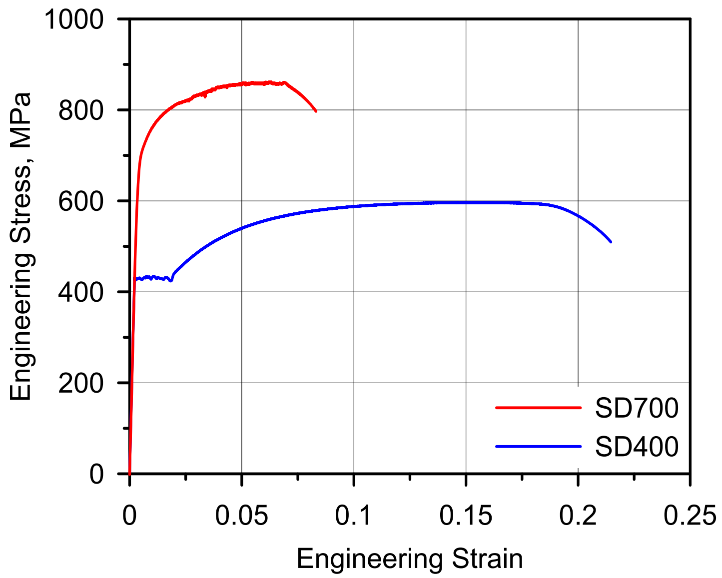


Figure 1.1 Stress-strain curve of normal and high strength rebar

The yield strength of rebar is limited by concrete design codes. The reason for this is that serviceability performance of reinforced concrete structures may decrease as far rebar spacing when yield strength of rebar increases (Lee et al 2010).

Table 1.1 Maximum yield strength of rebar in design codes

	Design codes	Flexure (MPa)	Shear (MPa)	Torsion (MPa)
Ultimate strength design	KDS 14 20 00	600	500	500
	ACI 318-19	690*)	420*)	420
	AASHTO LRFD 2020	690*)	690	690
Limit state design	KDS 24 14 21	600	600	600
	CSA A23.3-19	500	500	500
	EC2 2005	600	600	600

*) Non-seismic design

Yield strength of rebar is the important variable in serviceability provisions of design codes. In Korea, SD600 SD500 rebars are allowed in KDS 14 20 00 and KDS 24 14 21 design codes based on previous research (Kim et al., 2010). In the United States, Grade 100 (690) is permitted under ACI 318-19 and AASHTO LRFD 2017. To date, however, SD700 rebar is not allowed in the Korean design code. Provisions and material standards are different in each country's design codes. In order to include high strength rebar in design codes, research on applicability for code provisions on high strength rebar must be conducted taking into account the various design codes.

Table 1.2 Previous studies on the applicability of high strength rebar

f_y (MPa)	Research publication	Flexure provision	Serviceability provision
500 & 600	Gilbert R.I., 2007	○	○
	Peter H. Bischoff et al., 2009	○	○
	Jijun Tang et al., 2008	○	○
	Kim et al., 2010	○	○
700 & over	Lee et al., 2010	○	x
	Paul Zia et al., 2010	○	○
	Shahrooz et al., 2011	○	○
	Saif Aldabagh et al., 2018	○	○

Previous research (Kim et al., 2010) investigated applicability for code provisions related to flexure and serviceability provisions on SD500 and SD600 rebars. Other previous studies also conducted to investigate applicability for high strength rebars to design codes. For applying SD700 rebar to Korean design code, evaluation of the applicability of existing code provisions on SD700 is needed. Some previous research (Lee et al., 2010) has conducted to assess the applicability of code provisions related to flexure on SD700.

However, SD700 rebar has not yet been allowed under the Korean design codes. In order to incorporate SD700 rebar into Korean design codes, additional research is required. Specifically, investigation of the applicability of code provisions related to serviceability of SD700 is needed.

In this study, flexural tests of RC beam and one way slab reinforced with SD700 rebars were conducted to evaluate the applicability of code provisions related to serviceability such as crack width and deflection on SD700 rebar.

1.2. Research Objectives and Scope

This study aims to evaluate the applicability of existing Korean code provisions related to serviceability on SD700 rebar. Serviceability provisions such as crack width and deflection are covered in this study. The study has two research objectives. The first research objective is to evaluate the code provision on indirect control of crack width. The second research objective is to evaluate the code provision on indirect control of deflection.

This study consists of three main parts. First, a review of the literature on high strength rebar. Second, an experimental program. Finally, an applicability check for Korean design code on SD700 rebar. Indirect control of crack width and deflection of serviceability are the only design provisions covered in this study.

Testing was conducted on two flexural members, RC beam and one way slab. Specimens were reinforced with SD700 rebar considering maximum rebar spacing and minimum amount of reinforcement that is presented in design code to assess the applicability of the code provision related to indirect control crack width. Furthermore, different compressive strengths of concrete were used to observe the effect of concrete strength on performance of members with SD700 rebar. Test results were used for evaluation of crack width. For evaluation of deflection, test results were not used due to insufficient data. Instead, analysis method was used to evaluate deflection.

1.3. Outline

In chapter 1, research background, objectives, scope, and outline of this study are introduced.

In Chapter 2, basic concepts of high strength rebar and literature review of previous studies are presented. Also, limitations of the previous studies are discussed.

In chapter 3, definition of flexural members, mechanical difference of RC beam and one way slab and experimental program are presented. Design of test specimens, test procedure and test results are also included.

In chapter 4, evaluation of the applicability of code provisions related to indirect crack control and deflection on SD700 rebar is conducted. We consider existing Korean code provisions to assess their applicability for SD700 rebar. The result of test and analysis are used.

In chapter 5, results of evaluation for code provisions on SD700 rebar and conclusions of this study are summarized.

2. Literature Review

In existing Korean concrete design code, yield strength of longitudinal rebar is limited to 600 MPa that is SD600. In this chapter, concepts for necessity and problems of using high strength rebar are covered. Also, serviceability provisions dependent on yield strength of rebar are described. Previous studies on applicability of high strength rebar considering code provisions are presented.

2.1. High Strength Rebar

2.1.1. Necessity of high strength rebar

Reinforced concrete is a versatile composite and one of the most widely used materials in modern construction. Concrete is a relatively brittle material that is strong in compression but less so in tension. Unreinforced concrete is unsuitable for many engineering structures as it is relatively poor at withstanding stresses. To increase the overall strength of concrete, rebar can be embedded in concrete before it sets. This reinforcement resists tensile forces. By forming a strong bond together, the two materials are able to resist a variety of applied forces, effectively acting as a single structural element. Therefore, the use of high strength rebar in conjunction with high strength concrete is an effective combination for improved structural performance.

2.1.2. Advantages and problems of using high strength rebar

The use of high strength rebar provides several advantages. It can not only reduce the amount of reinforcement in the structural member but also conserves materials and construction costs and also helps reduce rebar congestion. This results in several benefits such as superior workmanship, improved durability and shorten construction time.

However important problem in using high strength rebar is that high strength rebar tends to have lower bond and ductility than that of normal strength rebar. The reason for this is that the material characteristics of high strength rebar are different from those of normal strength rebar.

2.2. Yield Strength of Rebar

2.2.1. Maximum yield strength of rebar in design codes

The yield strength of rebar and required amount of reinforcement are inversely proportional. One consequence of this is that rebar spacing increases as the required amount of reinforcement decreases. Increased spacing may lead to excessive crack and deflections of reinforced concrete members. In order to prevent this kind of failure, yield strength of rebar is limited in concrete design codes.

Table 1.1 presents the maximum yield strength of rebar in each design code. Flexural, shear and torsion reinforcement are included. Flexural reinforcement is considered in this study.

In United States, the maximum permitted yield strength of rebar is 690 MPa for flexural reinforcement in AASHTO LRFD 2020 and ACI 318-19 concrete design codes.

In Canadian design code CSA A23.3-19, the maximum yield strength of rebar is designated as 500 MPa. For Euro code 2 2005 and Korean design codes such as KDS 14 20 00, KDS 24 14 21, the maximum yield strength of rebar is 600 MPa.

2.2.2. Code provisions related to crack and deflection control

The yield strength of rebar is an important variable in code provisions related to crack and deflection control. Details of code provisions are different in each concrete design codes, respectively.

2.2.2.1 Indirect crack control

The spacing of reinforcement is limited to control cracking in design codes (KDS 14 20 00, ACI 318-19, AASHTO LRFD 2020). The rebar spacing is calculated in KDS 14 20 00 as Eq.2.1 where κ_{cr} denotes exposed conditions coefficient of rebar. It is 280 for dry condition or 210 for outside of dry conditions. Spacing of reinforcement is calculated by Eqs.2.2 and 2.3 in ACI 318-19, AASHTO LRFD 2020. Eqs.2.1 and 2.2 where f_s, c_c describe tensile stress in reinforcement at service load and clear cover of reinforcement. Eq.2.3 where γ_e, β_s, f_{ss}, d_c denote exposure factor, ratio of flexural strain, calculated tensile stress in non-prestressed reinforcement at the service limit state and distance from extreme tension fiber to center of the longitudinal rebar.

Exposure factor γ_e is 1 for class 1 exposure condition or 0.75 for class 2 exposure condition. Class 1 exposure condition applies when cracks can be tolerated due to reduced concerns of appearance and/or corrosion. Class 2 exposure condition applies to transverse design of segmental concrete box girders for any loads applied prior to attaining full nominal concrete strength and when there is increased concern of appearance and / or corrosion. Ratio of flexural strain β_s is calculated by Eq.2.4 where h is overall thickness or depth of the component.

$$s \le 375\left(\frac{\kappa_{cr}}{f_s}\right) - 2.5c_c \tag{2.1}$$

$$s \le 380\left(\frac{280}{f_s}\right) - 2.5c_c \tag{2.2}$$

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \tag{2.3}$$

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)} \tag{2.4}$$

If crack control is required, a minimum amount of bonded reinforcement is required to control cracking in areas where tension is expected. The amount may be extimated from equilibrium between the tensile force in concrete just before cracking and the tensile force in reinforcement at yielding or at a lower stress if necessary to limit the crack width. Required minimum amount of reinforcement is calculated by Eqs.2.5 and 2.6 to control cracking in design codes (KDS 24 14 21, EC2 2005). Eqs.2.5 and 2.6 where k_c , k , A_{ct} , f_{ct}, $f_{ct,eff}$ denote coefficients of stress distribution and non-uniform self-equilibrating, area of concrete within tensile zone and mean values of the tensile strength of the concrete effective at the time when the cracks may first be expected to occur. f_y is the specified yield strength for non-prestressed reinforcement. The coefficient of non-uniform self-equilibrating k is 1 for webs with $h \leq 300$ mm or flanges with widths less than 300 mm or 0.62 for webs with $h \leq 800$ mm intermediate values may be interpolated. The coefficient of stress distribution $k_c = 1$ for pure tension. For bending or bending combined with axial forces, it is calculated as the Eqs.2.7 and 2.9.

$$A_{s,\min} = k_c k A_{ct} \frac{f_{ct}}{f_s} \tag{2.5}$$

$$A_{s,\min} f_y = k_c k f_{ct,eff} A_{ct} \tag{2.6}$$

For rectangular sections and webs of box sections and T-sections:

$$k_c = 0.4\left[1 - \frac{\sigma_c}{k_1 (h/h^*) f_{ct,eff}}\right] \leq 1 \tag{2.7}$$

Where, σ_c and k_1 denote the mean stress of concrete and coefficient considering the effects of axial forces. Mean stress of the concrete is calculated by Eq.2.8 where N_{Ed} is the axial force at the serviceability limit state. It should be determined considering the characteristic value of prestress and axial forces under the relevant combination of actions. $h^* = h$ for $h < 1$ mm,

$h^* = 1$ m for $h \geq 1$ m. $k_1 = 1.5$ if N_{Ed} is a compressive force, $k_1 = \frac{2h^*}{3h}$ if N_{Ed} is a tensile force.

$$\sigma_c = \frac{N_{Ed}}{bh} \tag{2.8}$$

For flanges of box sections and T-sections:

$$k_c = 0.9 \frac{F_{cr}}{A_{ct} f_{ct,eff}} \geq 0.5 \tag{2.9}$$

Where, F_{cr} denotes the absolute value of the tensile force.

In Canadian design code CSA A23.3-19, quantity limiting distribution of flexural reinforcement is presented to control cracking that is calculated by Eq.2.10 where f_s, d_c and A describe tensile stress in reinforcement at service load, distance from extreme tension fiber to center of the longitudinal rebar and effective tension are of concrete.

$$z = f_s (d_c A)^{1/3} \tag{2.10}$$

2.2.2.2 Direct crack control

Crack width is calculated by Eq.2.11 in design codes (KDS 24 14 21 and EC2 2005). Eq.2.11 where $l_{\mathrm{r,max}}$, ε_{sm}, ε_{cm} denote maximum crack spacing, mean strain in the reinforcement and concrete that are calculated by Eqs.2.12, 2.15, 2.18 and 2.19. ε_{sm} is the under the relevant combination of loads, including the effect of imposed deformations and taking into account the effects of tension stiffening. Only the additional tensile strain beyond the state of zero strain of the concrete at the same level is considered.

$$\omega_k = l_{r,\max}(\varepsilon_{sm} - \varepsilon_{cm}) \tag{2.11}$$

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s - k_t \dfrac{f_{ct,eff}}{\rho_{p,eff}}(1 + \alpha_e \rho_{p,eff})}{E_s} \geq 0.6\frac{\sigma_s}{E_s} \tag{2.12}$$

Where, σ_s is the stress in the tension reinforcement assuming a cracked section. For pretensioned members, σ_s may be replaced by $\Delta\sigma_p$ for the stress variation in prestressing tendons from the state of zero strain of the concrete at the same level.

α_e is the ratio E_s / E_{cm}.

E_s is design value of modulus of elasticity of reinforcing steel.

E_{cm} is secant modulus of elasticity of concrete.

k_t is a factor dependent on the duration of the load. $k_t = 0.6$ for short term loading, $k_t = 0.4$ for long term loading.

$\rho_{p,eff}$ is calculated by following Eq.2.13.

$$\rho_{p,eff} = \frac{A_s + \xi_1^2 A_p^{'}}{A_{c,eff}} \tag{2.13}$$

Where, A_s is the cross-sectional area of reinforcement.

$A_p^{'}$ is the area of pre or post-tensioned tendons within $A_{c,eff}$.

$A_{c,eff}$ is the effective area of concrete in tension surrounding the reinforcement or prestressing tendons of depth, $h_{c,ef}$, where $h_{c,ef}$ is the lesser of $2.5(h-d)$, $(h-x)/3$ or $h/2$.

ξ_1 is the adjusted ratio of bond strength taking into account the difference in diameters of prestressing and reinforcing steel is defined in Eq.2.14. If only prestressing steel is used to control cracking $\xi_1 = \sqrt{\xi}$.

$$\xi_1 = \sqrt{\xi \frac{\phi_s}{\phi_p}} \tag{2.14}$$

Where ξ is ratio of bond strength of prestressing and reinforcing steel, according to Table 2.1.

ϕ_s is the largest bar diameter of reinforcing steel.

ϕ_p is the equivalent diameter of tendon.

Table 2.1 Ratio of bond strength, ξ, between tendons and reinforcing steel.

Prestressing steel	ξ		
	Pre-tensioned	Bonded, post-tensioned	
		$\le$ C50/60	$\ge$ C70/85
Smooth bars and wires	Not applicable	0.3	0.15
Stands	0.6	0.5	0.25
Intended wires	0.7	0.6	0.3
Ribbed bars	0.8	0.7	0.35

Note: For intermediate values between C50/60 and C70/85 interpolation may be used.

Maximum crack spacing is calculated depending on the following four conditions.

- In the first situation, where bonded reinforcement is fixed at reasonably close centers within the tension zone (spacing $\leq 5(c+\phi/2)$), the maximum final crack spacing is calculated by Eq.2.15. (see Figure 2.1)

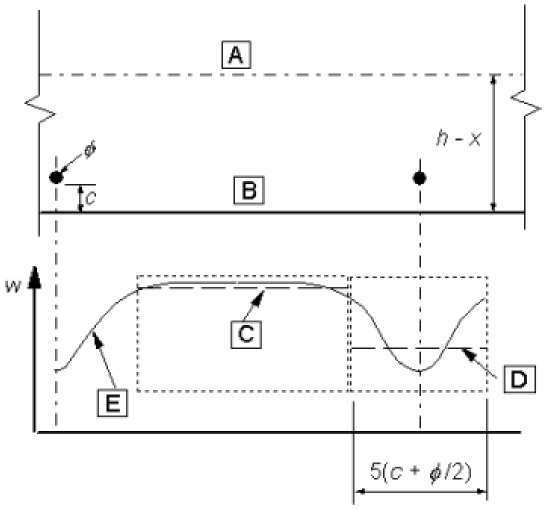


Figure 2.1 Crack width, *w*, at concrete surface relative to distance from bar

$$l_{r,\max} = 3.4c + \frac{0.425 k_1 k_2 \phi}{\rho_{p,eff}} \qquad (2.15)$$

Where ϕ is the bar diameter. Where a mixture of bar diameters is used in a section, an equivalent diameter, ϕ_{eq}, should be used. For a section with n_1 bars of diameter ϕ_1 and n_2 bars of diameter ϕ_2, the following Eq.2.16 should be used.

$$\phi_{eq} = \frac{n_1\phi_1^2 + n_2\phi_2^2}{n_1\phi_1 + n_2\phi_2} \qquad (2.16)$$

c is the cover to the longitudinal reinforcement.

k_1 is a coefficient which takes account of the bond properties of the bonded reinforcement. $k_1 = 0.8$ for high bond bars, $k_1 = 1.6$ for bars with an effectively plain surface (e.g. prestressing tendons).

k_2 is a coefficient which takes account of the distribution of strain. $k_2 = 0.5$ for bending, $k_2 = 1$ for pure tension, for cases of eccentric tension or for local areas, intermediate values of k_2 should be used which may be calculated from the relation Eq.2.17 where ε_1 is the greater than ε_2 is the lesser tensile strain at the boundaries of the section considered, assessed on the basis of a cracked section.

$$k_2 = (\varepsilon_1 + \varepsilon_2) / 2\varepsilon_1 \qquad (2.17)$$

- In the second situation, where the spacing of the bonded reinforcement exceeds $5(c + \phi / 2)$ (see Figure 2.1) or where there is no bonded reinforcement within the tension zone, an upper bound to the crack width may be found by assuming a maximum crack spacing (Eq.2.18).

$$l_{r,\max} = 1.3(h - x) \qquad (2.18)$$

- In the third situation, where the angle between the axes of principal stress and the direction of the reinforcement, for members reinforced in two orthogonal directions, is significant $(>15°)$, then the crack spacing $l_{r,\max}$ may be calculated from the following Eq.2.19.

$$l_{r,\max} = \frac{1}{\dfrac{\cos\theta}{l_{r,\max,y}} + \dfrac{\sin\theta}{l_{r,\max,z}}} \tag{2.19}$$

Where θ is the angle between the reinforcement in the y direction and the direction of the principal tensile stress.

$l_{r,\max,y}$, $l_{r,\max,z}$ are the crack spacings calculated in the y and z directions respectively.

- In the fourth situation, for walls subjected to early thermal contraction where the horizonal steel area, A_s does not fulfill the requirements of minimum reinforcement areas and where the bottom of the wall is restrained by a previously cast base, $l_{r,\max}$ may be assumed to be equal to 1.3 times the height of the wall.

Note: Where simplified methods of calculating crack width are used, they should be based on properties given in this standard or substantiated by tests.

2.2.2.3 Indirect deflection control

Members subjected to flexure shall be designed with adequate stiffness to limit deflections or deformations that adversely affect strength or serviceability of a structure. The minimum height of member is limited to control deflection in design codes (KDS 14 20 00, ACI 318-19, CSA A23.3-19). Minimum height is calculated as Eq.2.20 where l, f_y denotes span and yield strength of rebar.

$$h_{\min} = \frac{l}{16}\left(0.43 + \frac{f_y}{700}\right) \quad (2.20)$$

Table 2.2 presents minimum depth of member as designated in design codes that apply to one-way construction not supporting or attached to partitions or other construction likely to be damaged by large deflections, unless computation of deflection indicates a lesser depth can be used without adverse effects.

Table 2.2 Minimum depth of non-prestressed members

	Minimum depth, h			
	Simply supported	One end continuous	Both ends continuous	Cantilever
Solid one-way slab	$l/20$	$l/24$	$l/28$	$l/10$
Beams or ribbed one-way slabs	$l/16$	$l/18.5$	$l/21$	$l/8$

Notes:

1) This table gives traditional values that provide guidance for preliminary proportioning but are insufficient for beams or one-way slabs supporting partitions or other construction likely damaged by large deflections.
2) The values specified in this table are to be used directly for members with normal-density concrete where $\gamma_c > 2150$ kg/m^3 and the reinforcement is Grade 400. For other conditions, the values should be modified as follows:
   a) For structural low-density concrete and structural semi-low-density concrete, the values should be multiplied by

$(1.65-0.0003\gamma_c)$, but not less than 1, where γ_c is the density in kilograms per cubic meter.

b) For f_y other than 400 MPa, the values should be multiplied by $(0.43+f_y/700)$.

In design codes (AASHTO LRFD 2020, KDS 24 14 21 and EC2 2005), span/depth ratio is limited to control deflection. For reinforced concrete member, span to depth ratios are presented in Table 2.3 in AASHTO LRFD 2020, in which *S* is the slab span length and *L* is the span length, both in feet, and may be considered in the absence of other criteria.

Table 2.3 Traditional minimum depths for constant depth superstructures

<table>
<tr><th colspan="2" rowspan="1">Superstructure</th><th colspan="2">Minimum depth
(Including deck)

When variable depth members are used, values may be adjusted to account for changes in relative stiffness of positive and negative moment sections.</th></tr>
<tr><th>Material</th><th>Type</th><th>Simple spans</th><th>Continuous spans</th></tr>
<tr><td rowspan="4">Reinforced concrete</td><td>Slabs with main reinforcement parallel to traffic</td><td>$\frac{1.2(S+10)}{30}$</td><td>$\frac{S+10}{30}\geq 0.54$ ft</td></tr>
<tr><td>T-Beams</td><td>$0.070L$</td><td>$0.065L$</td></tr>
<tr><td>Box beams</td><td>$0.060L$</td><td>$0.055L$</td></tr>
<tr><td>Pedestrian structure beams</td><td>$0.035L$</td><td>$0.033L$</td></tr>
</table>

The limiting span/depth ratio is estimated using Eq.2.21.a, Eq.2.21.b in KDS 24 14 21, EC2 2005 and multiplying this by correction factors to allow for the type of reinforcement used and other variables. No allowance has been made for any pre-camber in the derivation of these equations.

$$\frac{l}{d}=K\left[11+1.5\sqrt{f_{ck}}\frac{\rho_0}{\rho}+3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho}-1\right)^{3/2}\right] \quad \text{if} \quad \rho \le \rho_0 \qquad (2.21.a)$$

$$\frac{l}{d}=K\left[11+1.5\sqrt{f_{ck}}\frac{\rho_0}{\rho-\rho'}+\frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_0}}\right] \quad \text{if} \quad \rho > \rho_0 \qquad (2.21.b)$$

Where l/d is the limit span/depth.

K is the factor to take into account the different structural systems, Table 2.4.

ρ_0 is the reference reinforcement ratio, $(\rho_0=\sqrt{f_{ck}}10^{-3})$.

ρ is the required tension reinforcement ratio at mid span to resist the moment due to design loads (at support for cantilevers).

ρ' is the required compression reinforcement ratio at mid span to resist the moment due to design loads (at support for cantilevers).

Eqs.2.21.a and b have been derived on the assumption that the steel stress, under the appropriate design load at serviceability limit state at a cracked section at the mid span of a beam or slab or at the support of a cantilever, is 310 MPa.

For flanged sections where ratio of the flange breadth to the rib breadth exceeds 3, the values of l/d given by Eq.2.21 should be multiplied 0.8.

For beams and slabs, other than flat slabs, with spans exceeding 7 m, which support partitions liable to be damaged by excessive deflections, the values of l/d given by Eq.2.21 should be multiplied by $7/l_{eff}$ (l_{eff} in meters).

For flat slabs where the greater span exceeds 8.5 m and which support partitions liable to be damaged by excessive deflections, the values of l/d given by Eq.2.21 should be multiplied by $8.5/l_{eff}$ (l_{eff} in meters).

Table 2.4 Basic ratios of span/depth for reinforced concrete members without axial compression

Structural system	K	Concrete highly stressed $\rho = 1.5\%$	Concrete lightly stressed $\rho = 0.5\%$
Simply supported beam, one or two way spanning simply supported slab	1	14	20
End span of continuous beam or one-way continuous slab or two-way spanning slab continuous over one long side	1.3	18	26
Interior span of beam or one-way or two-way spanning slab	1.5	20	30
Slab supported on columns without beams (flat slab) (based on longer span)	1.2	17	24
Cantilever	0.4	6	8

Note 1: The values given have been chosen to be generally conservative and calculation may frequently show that thinner members are possible.
Note 2: For two-way spanning slabs, the check should be carried out on the basis of the shorter span. For flat slabs the longer span should be taken.
Note 3: The limits given for flat slabs correspond to a less severe limitation than a mid span deflection of span/250 relative to the columns.

2.2.2.4 Direct deflection control

When deflections are to computed, deflections that occur immediately on application of load shall be computed by methods or formulas for elastic deflections, taking into consideration the effects of cracking and reinforcement on member stiffness (CSA A23.3-19). Deflection shall be computed using elastic deflection equation as Branson equation Eq.2.22 in design codes (KDS 14 20 00, AASHTO LRFD 2020 and CSA A23.3-19). The effective moment of inertia procedure described in the code and developed in Branson (1965) was selected as being sufficiently accurate to estimated deflections. Eq.2.22 where the effective moment of inertia, I_e, was developed to provide a transition between the upper and lower bounds of I_g and I_{cr} as a function of the ratio M_{cr} / M_a.

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \le I_g \qquad (2.22)$$

Where M_{cr} is the cracking moment, It is defines as Eq.2.23.

M_a is maximum moment in member due to service loads at stage deflection.

I_g is moment of inertia of gross concrete section about centroidal axis.

I_{cr} is moment of inertia of cracked section transformed to concrete.

For non-prestressed members, effective moment of inertia, I_e shall not be greater than I_g.

$$M_{cr} = \frac{f_r I_g}{Y_t} \qquad (2.23)$$

Where f_r, y_t denotes modulus of rupture of concrete and distance from centroidal axis of gross section, neglecting reinforcement, to tension face.

Modulus of rupture of concrete is defined as Eq.2.24 depending on density of concrete.

$$f_r = 0.6\lambda\sqrt{f_{ck}} \qquad (2.24)$$

Where λ, f_{ck} denotes factor to account for low-density concrete and compressive strength of concrete.

$\lambda = 1$ for normal density concrete.

$\lambda = 0.85$ for structural semi low density concrete in which all the fine aggregate is natural sand.

$\lambda = 0.75$ for structural low density concrete in which none of the fine aggregate is natural sand.

The effective moment of inertia approximation, developed by Bischoff (2005), has been shown to result in calculated deflections that have sufficient accuracy for a wide range of reinforcement ratios (Bischoff and Scanlon 2007). M_{cr} is multiplied by two-thirds to consider restraint that can reduce the effective cracking moment as well as to account for reduced tensile strength of concrete during construction that can lead to cracking that later affects service deflections (Scanlon and Bischoff 2008). For non-prestressed members, unless obtained by a more comprehensive analysis, effective moment of inertia, I_e, shall be calculated by Bischoff and Scanlon equation accordance with Table 2.5 in ACI 318-19.

Table 2.5 Effective moment of inertia, I_e

Service moment	Effective moment of inertia, I_e, in^4
$M_a \leq (2/3)M_{cr}$	I_g
$M_a > (2/3)M_{cr}$	$\dfrac{I_{cr}}{1-\left(\dfrac{(2/3)M_{cr}}{M_a}\right)^2\left(1-\dfrac{I_{cr}}{I_g}\right)}$

Maximum permissible computed deflections are specified as Table 2.6 in design codes (KDS 14 20 00, ACI 318-19 and CSA A23.3-19). It should be noted that the limitations given in Table 2.6 relate only to supported or attached nonstructural elements. For those structures in which structural members are likely to be affected by deflection or deformation of members to which they are attached in such a manner as to affect adversely the strength of the structure, these deflections and the resulting forces should be considered explicitly in the analysis and design of the structures as required by 24.2.1 (ACI 209R). When time dependent deflections are calculated, the portion of the deflection before attachment of the nonstructural elements may be deducted ACI318-19.

Table 2.6 Maximum permissible calculated deflections

Member	Condition		Deflection to be considered	Deflection limitation
Flat roofs	Not supporting or attached to nonstructural elements likely to be damaged by large deflections		Immediate deflection due to maximum of L_r ,S and R	$l/180$[1]
Floors			Immediate deflection due to specified live load, L	$l/360$
Roof or floors	Supporting or attached nonstructural elements	Likely to be damaged by large deflections	That part of the total deflection occurring after attachment of nonstructural elements, which is the sum of the time dependent deflection due to all sustained loads and the immediate deflection due to any additional live load [2]	$l/480$[3]
		Not likely to be damaged by large deflections		$l/240$[4]

[1] Limit not intended to safeguard against ponding. Ponding shall be checked by calculations of deflection, including added deflections due to ponded water, and considering time dependent effects of sustained loads, camber, construction tolerances, and reliability of provisions for drainage.

[2] Time dependent deflection shall be calculated in accordance with Table 2.6, but shall be permitted to be reduced by amount of deflection calculated to occur before attachment of nonstructural elements. This amount shall be calculated on basis of accepted engineering data relating to time deflection characteristics of members similar to those being considered.

[3] Limit shall be permitted to be exceeded if measures are taken to prevent damage to supported or attached elements.
[4] Limit shall not exceed tolerance provided for nonstructural elements.

Members which are not expected to be loaded above the level which would cause the tensile strength of the concrete to be exceeded anywhere within the member should be considered to be uncracked. Checking deflection by calculation is determined by Eq.2.25 in design codes (KDS 24 14 21 and EC2 2005).

$$\Delta_e = \zeta\Delta_{crack} + (1-\zeta)\Delta_{uncrack} \tag{2.25}$$

Where Δ_e is the deformation parameter considered which may be, for example, a strain, a curvature, or a rotation. (As a simplification, Δ_e may also be taken as a deflection.

$\Delta_{crack}, \Delta_{uncrack}$ are the values of the parameter calculated for the fully cracked and uncracked conditions, respectively.

ζ is a distribution coefficient (allowing for tensioning stiffening at a section) given by Eq.2.26. $\zeta = 0$ for uncracked section.

$$\zeta = 1 - \beta\left(\frac{\sigma_{sr}}{\sigma_s}\right)^2 \tag{2.26}$$

Where β is a coefficient taking account of the influence of the duration of the loading or of repeated loading on the average strain. $\beta = 1$ for a single short term loading, $\beta = 0.5$ for sustained loads or many cycles of repeated loading

σ_s is the stress in the tension reinforcement calculated on the basis of a cracked section.

σ_{sr} is the stress in the tension reinforcement calculated on the basis of a cracked section under the loading conditions causing first cracking.

Note: σ_{sr} / σ_s may be replaced by M_{cr} / M for flexure or N_{cr} / N for pure tension, where M_{cr} is the cracking moment and N_{cr} is the cracking force.

2.3. Previous Studies

2.3.1. Kim, Jin-Keun et al (2010)

To evaluate the applicability for existing Korean concrete design code on SD500 and SD600 as 500 MPa and 600 MPa high strength rebars, Kim, Jin-Keun et al (2010) performed the flexural test of 12 RC rectangular beams with 4 points loading. The code provisions related to flexure and serviceability were checked in this study. Test result showed that SD500 and SD600 rebars are applicable for code provisions related to flexure and serviceability of existing Korean design codes.

Moreover, minimum height of member was inspected by analysis for check the control deflection. Based on this study, SD500 and SD600 rebars were allowed in existing Korean concrete design code. The maximum yield strength of rebar is 600 MPa in existing Korean concrete design code.

For more than 600 MPa high strength rebar applying to existing Korean concrete design code, evaluation of applicability for code provisions on that is needed. If more than 600 MPa high strength rebars are not applicable for code provisions. Revision is needed in existing Korean concrete design code for applying more than 600 MPa high strength rebar.

2.3.2. Lee, Jae-Hun et al (2010)

Lee, Jae-Hun et al (2010) conducted the research for more than 600 MPa high strength rebar applying existing Korean concrete design code and performed the flexural test of 14 RC rectangular beams with 4 points loading to evaluate the flexural performance of RC beam reinforced with SD700 as 700 MPa high strength rebar. From test results, flexural strength was higher than nominal flexural strength. And crack width was increased.

And minimum reinforcement ratio was inspected by analysis. In this study SD700 rebars was evaluated for applying existing Korean concrete design code considering flexural performance of RC beams with SD700. But SD700 rebar has not allowed in design codes.

Additional research on SD700 rebar is needed for applying existing Korean design code. For using high strength rebar that effects the serviceability of members. Samely, evaluation of applicability for code provisions related to serviceability on SD700 is necessary.

2.3.3. Limitations of previous studies

In experimental researches for high strength rebar, RC beams which had various rebar amount were tested. In Korea, SD500 and SD600 rebars were allowed in existing concrete design codes based on previous study such as Kim, Jin-Keun et al (2010). Research of SD700 rebar also conducted by Lee, Jae-Hun et al (2010) for applying existing Korean concrete design code. But additional research is needed for applying design code.

For using high strength rebar, required rebar amount is decreased as yield strength of rebar increases. It is reason of excessive crack width and deflection of reinforced concrete members. But serviceability was not covered in previous research of SD700 rebar. Also, only beam was selected and evaluated from flexural members.

In order to overcome above mentioned limitations of previous researches, evaluation of applicability for code provisions related to serviceability such as crack and deflection on SD700 rebar is needed. In this study, flexural tests were performed on RC beams and one way slabs reinforced with SD700 rebars. For checking crack and deflection control, minimum rebar amount applied to specimens considering existing Korean concrete design code.

3. Experimental Program

This chapter covers the details of the flexural test. The test was designed to overcome the limitations of previous studies as mentioned in previous chapter. The flexural test was performed on RC beam and one way slabs. Below, the test procedure and results are described. Code provisions related to crack and deflection are then evaluated on test specimens.

3.1. Flexural Members

3.1.1. Beam

A beam is a structural element that is capable of withstanding load primarily by resisting bending. The bending force induced into the material of the beam as a result of the external loads, won weight, span and external reactions to these loads is called a bending moment. Beams are traditionally descriptions of building or civil engineering structural elements. Beams generally carry vertical gravitational forces but can also be used to carry horizontal loads. The loads carried by a beam are transferred to columns, walls, or girders, which then transfer the force to adjacent structural compression members. Beams are characterized by their profile (the shape of their cross section), their length and their material.

3.1.2. Slab

Structural concrete slabs are constructed to provide flat surfaces, usually horizontal, in building floors, roofs, bridges, and other types of structures. The slab may be supported by walls, by reinforced concrete beams usually cast monolithically with the slab, by structural steel beams, by columns, or by the ground. The depth of a slab is usually very small compared to its span. Concrete slab is a common structural element of modern building. It is usually horizontal and has smaller thickness comparative to its span. In general, slabs are classified as being one way or two way. Slabs that primarily deflect in one direction are referred to as one way slabs. When slabs are supported by columns arranged generally in rows so that the slabs can deflect in two directions they are usually referred to as two way slabs.

3.1.3. Difference between beam and slab

Beam is a linear structural element where perpendicular loads known as flex load are applied along with the axis. Slab is a flexural component that distributes the load horizontally to one or more directions within a single plane.

A beam is the most common example of a structural element in bending. While the resistance to bending of a slab is similar to that of a beam, it differs from that of a comparable series of independent beams in continuity in both directions.

Beam is the most direct solution to the most common structural problems of transferring horizontal loads of gravity to the load elements. A slab is used when concentrated loads result in a perpendicular bending located in the first direction of extension causing torsion in the slab.

3.2. Flexural Test

3.2.1. Test variables

Flexural test variables of this study are presented in Figure 3.1. To evaluate the applicability for code provisions related to crack and deflection on SD700, three types of compressive strength of concrete were chosen for the test specimens. Furthermore, SD 700 as 700 MPa high strength rebars were used to test specimens considering minimum amount of reinforcement that is presented in existing concrete design codes.

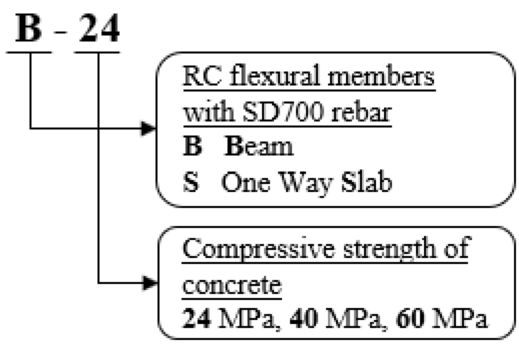


Figure 3.1 Designation of the test specimen group

Tests were conducted to evaluate the performance of RC beams and one way slabs reinforced with SD700 rebars. High strength concrete as well as normal strength concrete were included since high strength rebar is usually used with high strength concrete.

3.2.2. Design of test specimens

Figure 3.1 presents the flexural test variables for this study. Three types of compressive strength concrete were used for the test specimens in order to evaluate the applicability for code provisions related to crack and deflection on SD700. Additionally, SD 700 as 700 MPa high strength rebars were used to test the specimens based on the minimum amount of reinforcement needed to meet rebar yield strength requirements described in existing concrete design code. Test specimens are shown in Figure 3.2. Two group specimens that are three beams and three slabs were tested. The test specimen details are presented in Table 3.1.

Table 3.1 Test specimen details

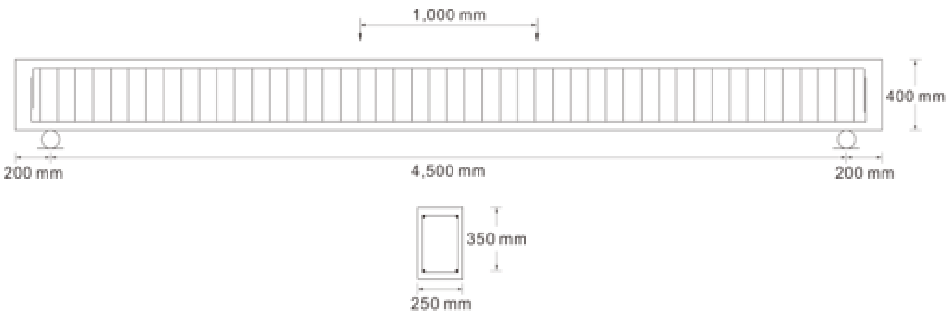
ID	f_{ck} (MPa)	f_y (MPa)	ρ	Tensile rebar	Comp- -ressive rebar	$B \times H \times L$ (mm)	M_n (kNm)
B-24	24	700	0.0029	2-D13	2-D13	250x400 x4500	60
B-40	40						63
B-60	60						65
S-24	24	700	0.0066	5-D13	2-D13	800x160 x2200	49
S-40	40						53
S-60	60						55

(1) Beam

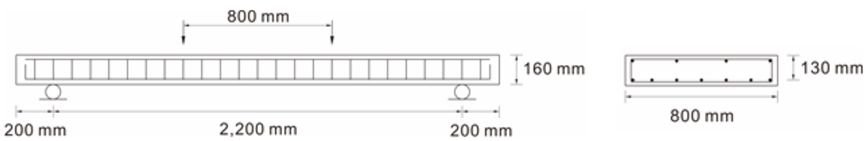
The height is designed by calculating Eq.2.20. Accordingly, span is designed as $L = 4500$ mm. The amount of tensile reinforcement is calculated by equation of minimum tensile rebar amount based on flexural provision of KDS 14 20 00 and KDS 24 14 21. Two D13 SD700 rebars were used for compressive reinforcement. For shear reinforcement, D10 rebars were used by spacing 100 mm to prevent flexural and shear failure.

(2) One way slab

The height is calculated by Eq.2.20. Accordingly, span is designed as $L = 2500$ mm. The amount of tensile rebar is designed in the same way as the beam according to Korean concrete design codes. Compressive and shear reinforcements are planned same as the beam.



(a)



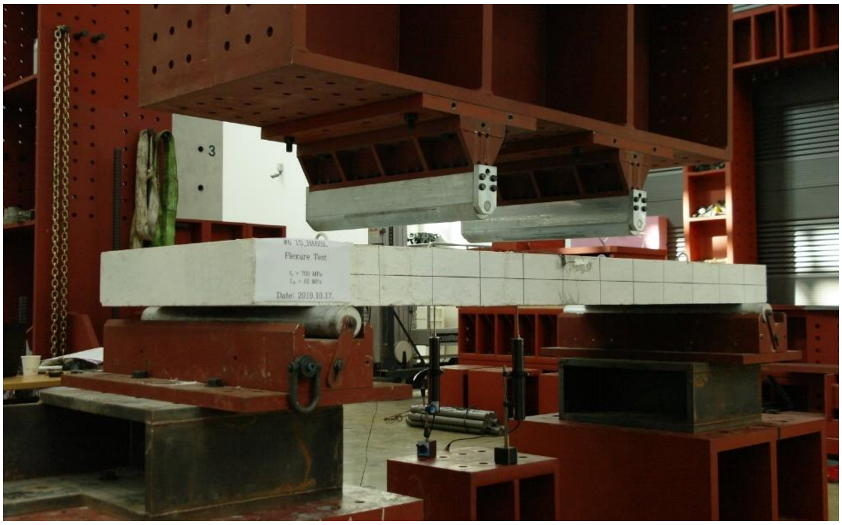
(b)

Figure 3.2 Test specimens; (a) RC beam; (b) One way slab

3.2.3. Test procedures

Apparatus at the Extreme Performance Testing Center at Seoul National University was used to conduct the experiment used for this study. Figure 3.3 presents images of the test configuration. Testing conditions were simply supported with $\varnothing 80$ mm roller at both ends and four points bending by displacement control. Load span was 1000 mm $(a/d=5)$, loading rate was 2 mm/min. Termination occurred at crushing of concrete.

(a)



(b)

Figure 3.3 Test setup; (a) RC beam; (b) One way slab

Data type and measuring tools are presented in Table 3.2. For one way slab, strain gauge was not installed on compressive rebars. Figure 3.4 shows the location of measuring tools.

Table 3.2 Data acquisition

Data type	Measuring tool	Location of measuring	RC beam	One way slab
			Number	
Tensile rebar strain	Strain gauge	Center of tensile rebar	4	6
Compressive rebar strain	Strain gauge	Center of compressive rebar	2	2
Concrete strain	Strain gauge	Center of upper side of compressive part of concrete	2	-
Deflection	LVDT	Center of beam lower side	2	2
Load	Actuator	-	-	-
Crack width	Light scale loupe	Load span	-	-

In Table 3.2, Linear Variable Differential Transformers (LVDT) are used to measure displacement. An LVDT measures displacement by associating a specific signal value for any given position of the core. This association of a signal value to a position occurs through electromagnetic coupling of an AC excitation signal on the primary winding to the core and back to the secondary windings.

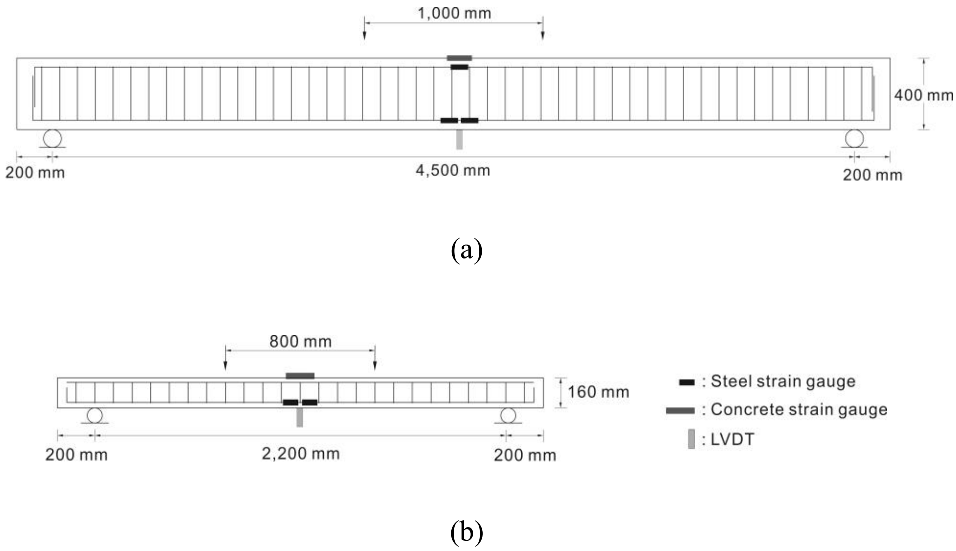


Figure 3.4 Location of measuring; (a) RC beam; (b) One way slab

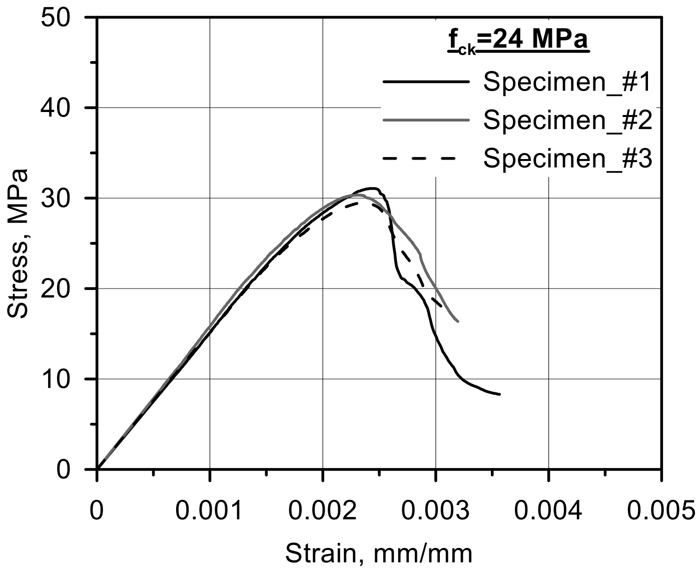
3.2.4. Test results

3.2.4.1 Material test

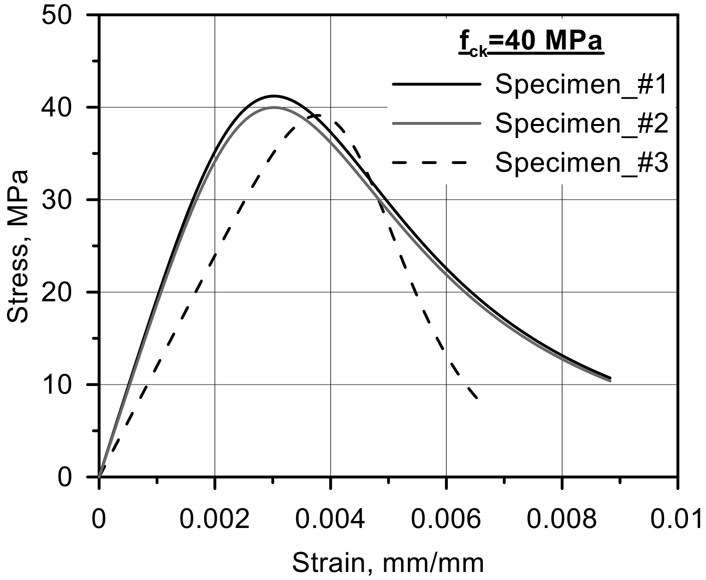
Compressive strength of concrete: Three specimens per each compressive strength of concrete, f_{ck} were tested following ASTM C39/C39M-16. Test results of each group are listed in Table 3.3. Stress- strain curves of concrete are shown in Figure 3.5 by each compressive strength of concrete. As shown in Figure 3.5, similar compressive strengths of concrete presented for each group.

Table 3.3 Material test result - Compressive strength of concrete

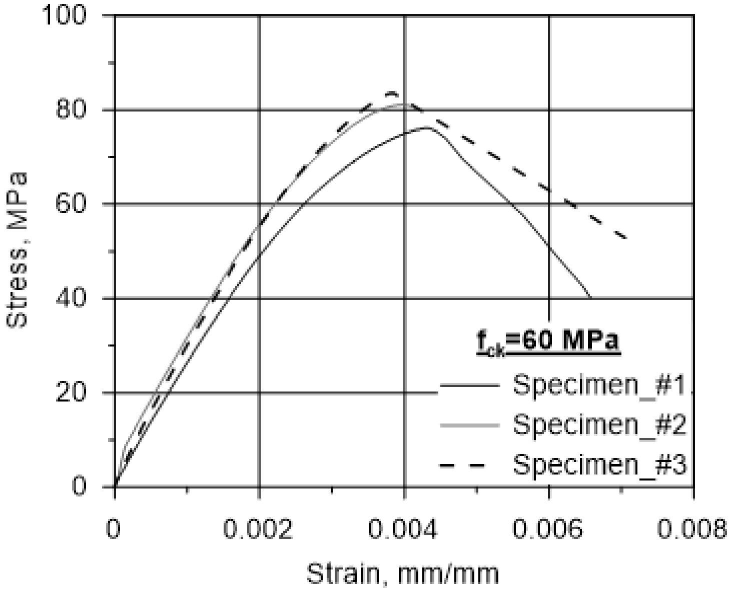
Test specimen number	$f_{ck}=24$ MPa		$f_{ck}=40$ MPa		$f_{ck}=60$ MPa	
	$f_c^{'}$ (MPa)	E_c (MPa)	$f_c^{'}$ (MPa)	E_c (MPa)	$f_c^{'}$ (MPa)	E_c (MPa)
#1	30.2	15054	41.2	19403	76.1	-
#2	29.4	15710	40	18829	81.1	23615
#3	28.6	15232	39.1	11996	83.5	24065
Average	29.4	15332	40.6	19116	80.2	23840
$f_c^{'}/f_{ck}$	1.22		1.01		1.33	



(a)



(b)



(c)

Figure 3.5 Stress-strain curve for concrete; (a) $f_{ck} = 24$ MPa; (b) $f_{ck} = 40$ MPa; (c) $f_{ck} = 60$ MPa

Strength of reinforcing steel: Three specimens of SD700 D13 rebars were tested following ASTM A370-18. Test results are shown in Table 3.4. Figure 3.6 shows similar strength of reinforcing steel for each specimen.

Table 3.4 Material test result - Strength of reinforcing steel

	SD700-D13			
	#1	#2	#3	Average
Yield strength, f_y (MPa)	725	796.2	727	749.4
Tensile strength, f_s (MPa)	869	926.5	899.9	898.5
f_s / f_y	1.24	1.16	1.24	1.21
KS standard, f_s / f_y	1.08			
Elastic modulus, E_s (MPa)	200000			

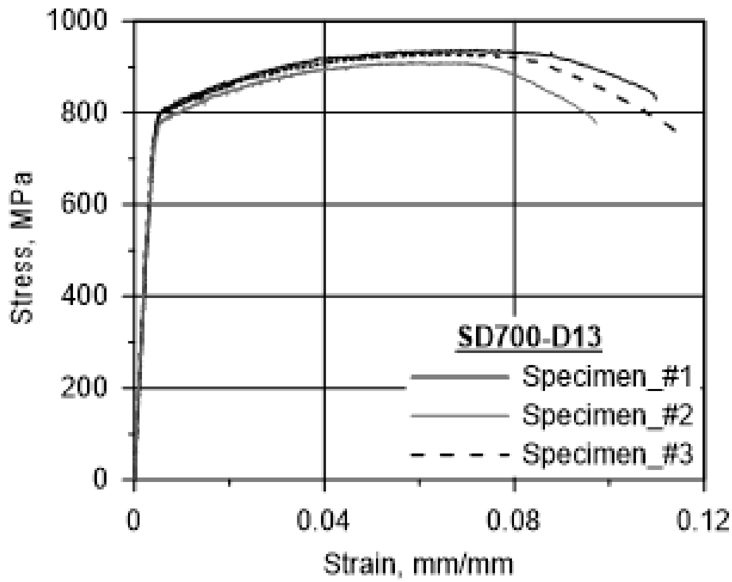


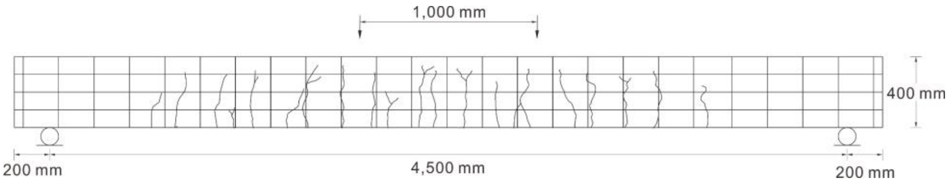
Figure 3.6 Stress-strain curve for reinforcing steel

3.2.4.2 Flexural test

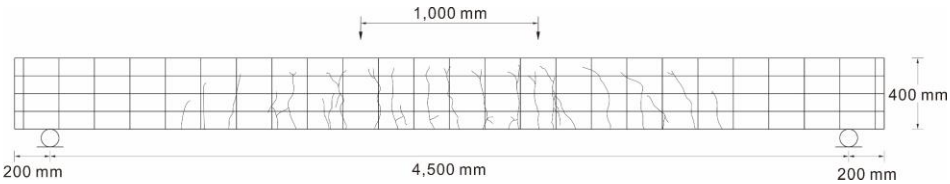
Flexural failure occurred in all specimens. Flexural test results are shown in Table 3.5. Crack patterns of RC beam and one way slab at failure are presented in Figures 3.7 and 3.8. The test results demonstrate the validity of the flexural test, indicating that the results data can be used for evaluation of applicability of code provisions related to crack and deflection on SD700 rebar.

Table 3.5 Flexural test result

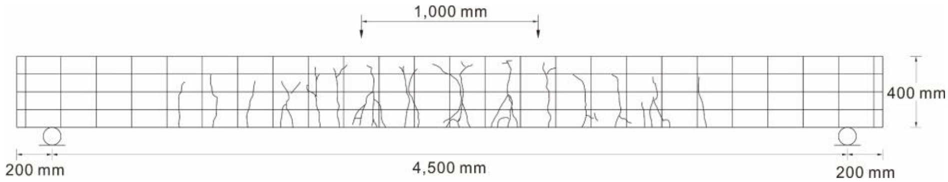
ID	Flexural strength (kN)	Ductility ratio	Failure mode
B-24	78.4	2.17	Flexure
B-40	73.7	2.67	
B-60	83.3	3.13	
S-24	165	2.21	Flexure
S-40	178.7	3.25	
S-60	196.1	3.89	



(a)

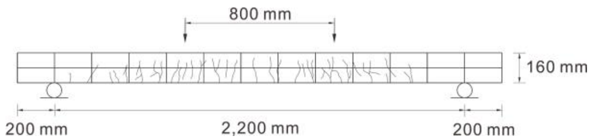


(b)

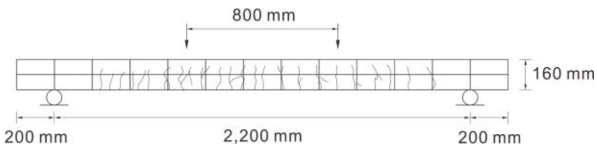


(c)

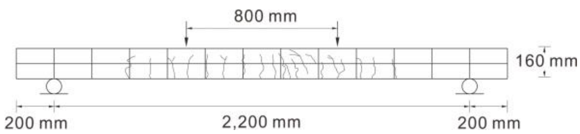
Figure 3.7 Crack pattern at failure - RC beam; (a) B24; (b) B40; (c) B60



(a)



(b)



(c)

Figure 3.8 Crack pattern at failure – One way slab; (a) S24; (b) S40; (c) S60

4. Applicability Check for KDS

In this chapter, the applicability for code provisions related to crack and deflection on SD700 rebar are discussed. Test result are used for evaluation of applicability for crack width. Additionally, sectional analysis is performed to evaluate deflection.

4.1. Serviceability Provisions

4.1.1. Crack width

When the flexural strength and cross section are the same, using higher steel yield strength decreases the amount of reinforcement. As the reduction in reinforcement increases the rebar spacing, crack width also increase as shown in Figure 4.1. Therefore, it is necessary to review crack width in terms of yield strength.

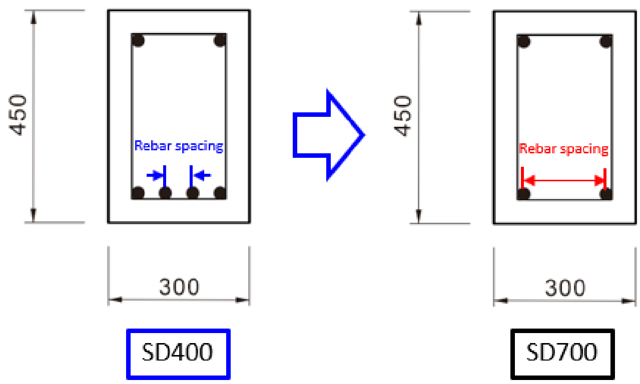


Figure 4.1 Rebar spacing for using SD400 and SD700 rebar

In KDS 14 20 00, crack width is controlled by the flexural rebar spacing that is calculated by Eq.2.1, which is presented in chapter 2 with variations depending on environmental condition. It prevents the excessive crack width due to reduction of reinforcement.

$$s \leq 375\left(\frac{\kappa_{cr}}{f_s}\right) - 2.5c_c \tag{2.1}$$

where κ_{cr} denotes exposed conditions coefficient of rebar. It is 280 for dry conditions or 210 for outside of dry conditions. And f_s , c_c describe tensile stress in reinforcement at service load and clear cover of reinforcement.

Allowable crack width is presented in KDS 14 20 00 depending on environment conditions as shown in Table 4.1. Crack width of specimens reinforced with SD700 was evaluated based on the presented allowable values. Flexural member with rebar spacing conforming to the design code presented with crack width lower than the allowable crack width.

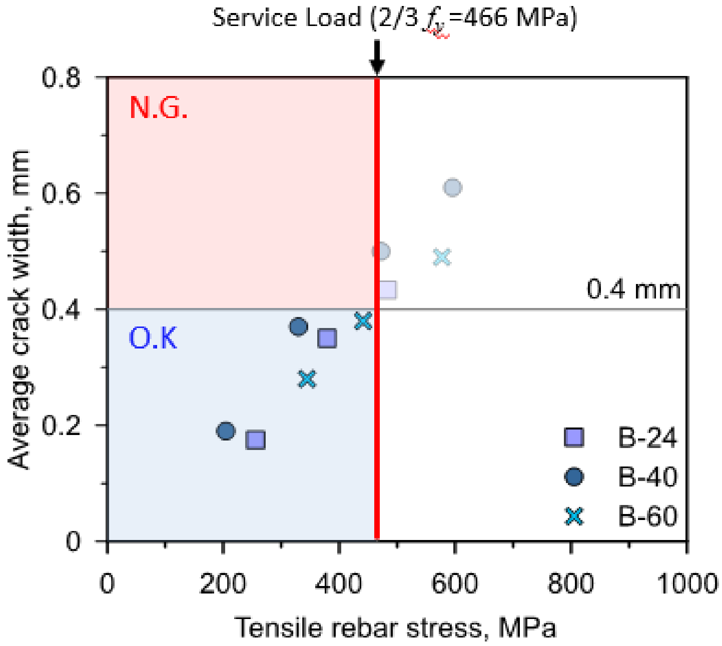
Table 4.1 Allowable crack width of reinforced concrete structures in KDS

Steel type	Environmental conditions for corrosion of steel			
	Dry	Moist	Corrosiveness	High corrosivity
Rebar	The larger of 0.4 mm and $0.006c_c$	The larger of 0.3 mm and $0.005c_c$	The larger of 0.3 mm and $0.004c_c$	The larger of 0.3 mm and $0.0035c_c$
Tendon	The larger of 0.2 mm and $0.005c_c$	The larger of 0.2 mm and $0.004c_c$	-	-

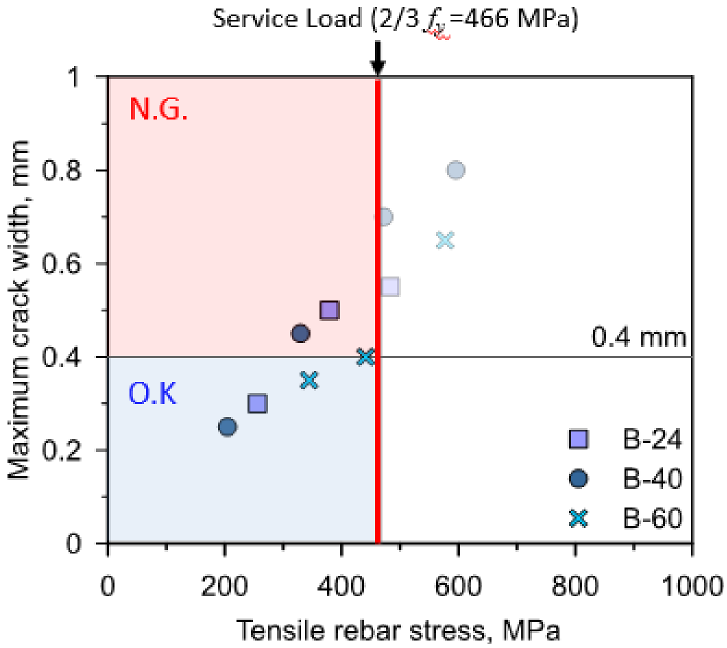
Where, c_c is clear cover of reinforcement

In dry environment conditions, flexural members have maximum rebar spacing. Test specimens have rebar spacing satisfied in dry environmental condition. Specimens reinforced with SD700 were examined for crack width exceeding the allowable width 0.4 mm.

Test results are shown in Figure 4.2 and Figure 4.3. The results illustrate that crack width of RC beams and one way slabs are similar or less than the allowable crack width specified in KDS 14 20 00 (see Table 4.1). Therefore, test results satisfied the allowable crack width under service load.

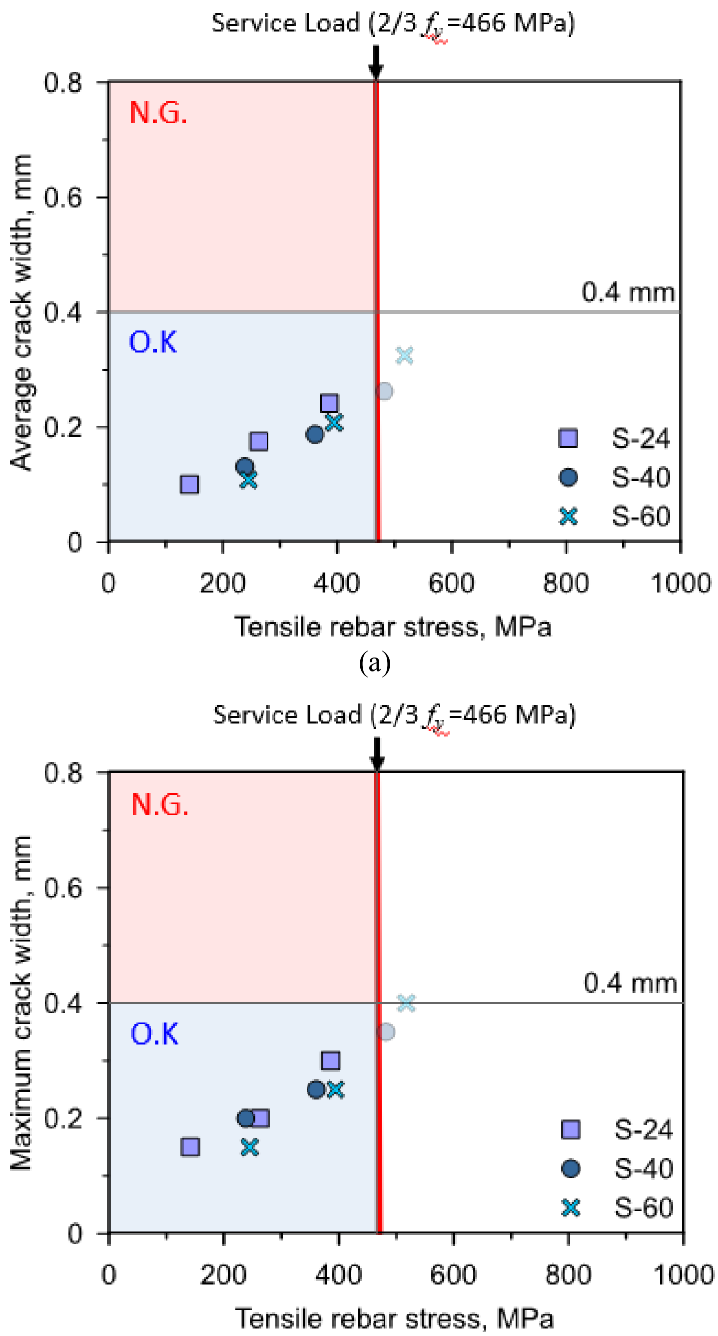


(a)



(b)

Figure 4.2 Crack width of RC beam; (a) Average; (b) Maximum



(a)

(b)

Figure 4.3 Crack width of one way slab; (a) Average; (b) Maximum

4.1.2. Deflection

When the flexural strength and cross section are the same, using higher steel yield strength decreases the amount of necessary reinforcement. Reduction in reinforcement will reduce the stiffness, which can cause excessive deflection as shown in Figure 4.4. Therefore, review of deflection is necessary in terms of yield strength.

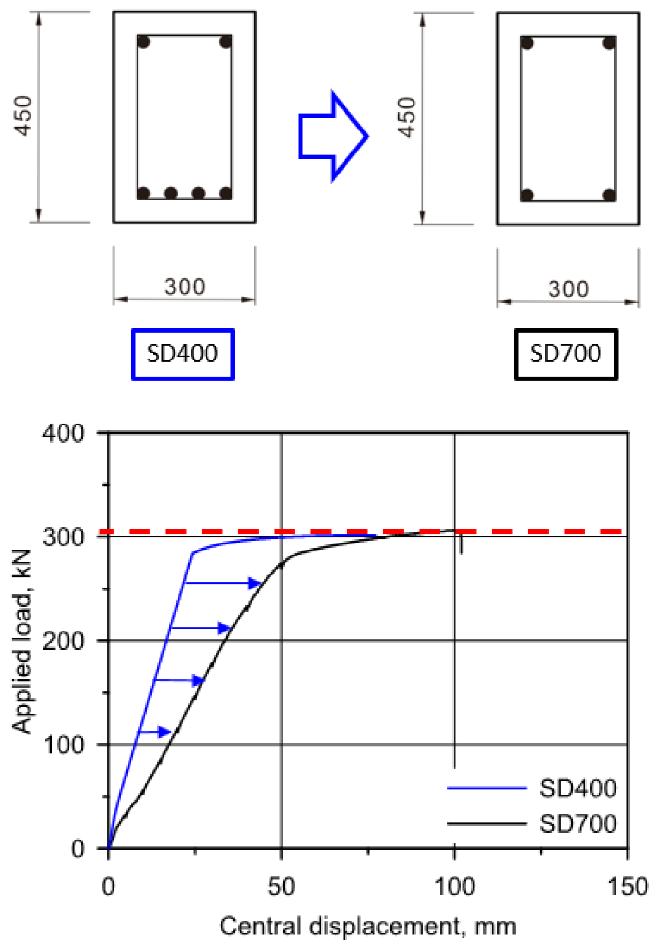


Figure 4.4 Stiffness for using SD400 and SD700 rebar

In KDS 14 20 00, deflection is controlled by minimum height of the member that prevents excessive deflection leading to a reduction of rebar amount. So, minimum height should be evaluated for applicability of deflection on SD700 rebar. The minimum height is specified in Table 2.2 in chapter 2. The calculation is applicable to one way construction not supporting or attached to partitions or other construction likely to be damaged by large deflections, unless computation or deflection indicates a lesser height can be used without adverse effects.

The evaluation method compares effective stiffness of members reinforced with SD400 and SD700 rebars with each minimum heights and having same flexural strengths. However, a problem is the absence of specimens reinforced with SD400 rebar designed with minimum height provision and same flexural strength. So, the effective stiffness for the minimum height of specimens reinforced with SD400 and SD700 is calculated through sectional analysis at various flexural strengths. The minimum height is calculated by Eq 2.20 based on yield strength of rebar.

$$h_{\min} = \frac{l}{16}\left(0.43 + \frac{f_y}{700}\right) \tag{2.20}$$

Where l, f_y denotes span and yield strength of rebar.

Calculation of effective stiffness: (EI_e).

Dimension ($B \times H \times L$):

RC beam: SD400→ 300x**313**x5000 mm, SD700→ 300x**450**x5000 mm

One way slab: SD400→ 800x**110**x2200 mm, SD700→ 800x**220**x2200 mm

Method for calculated effective stiffness (EI_e):

Effective stiffness was calculated using the Eq 4.1 under service load. After determining rebar ratio depending on flexural strength, the curvature was calculated by Eq 4.4. Neutral axis and curvature under service load are shown in Figures 4.5 and 4.6.

$$EI_e = \frac{M_S}{\phi_s} \tag{4.1}$$

Where M_s, ϕ_s denotes flexural moment at section and curvature under service load.

Figure 4.5 shows the neutral axis under service load that is calculated by Eqs 4.2 and 4.3.

$$kd = \frac{Q_x}{A} \tag{4.2}$$

$$k = -(\rho + \rho')n + \sqrt{(\rho + \rho')^2 n^2 + 2(\rho + \rho' d'/d)n} \tag{4.3}$$

Where ρ , ρ' denotes reinforcement ration for tensile and compression reinforcement. n means number of rebar. d is effective depth.

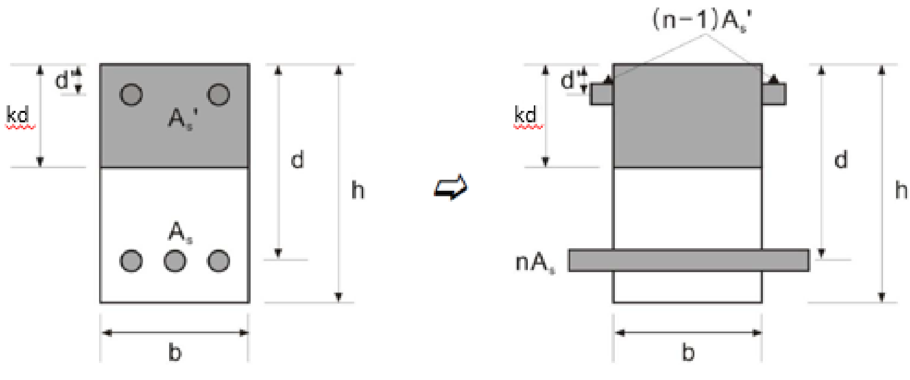


Figure 4.5 Neutral axis (kd) under service load

Figure 4.6 shows the curvature under service load that is calculated from proportion as described in Eq.4.4.

$$\phi_s = \frac{\varepsilon_c}{kd} = \frac{\varepsilon_s}{d - kd} \tag{4.4}$$

Where $\varepsilon_c, \varepsilon_s$ denote the compressive strain in the concrete and strain in reinforcement.

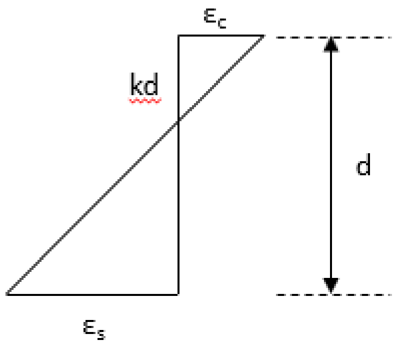
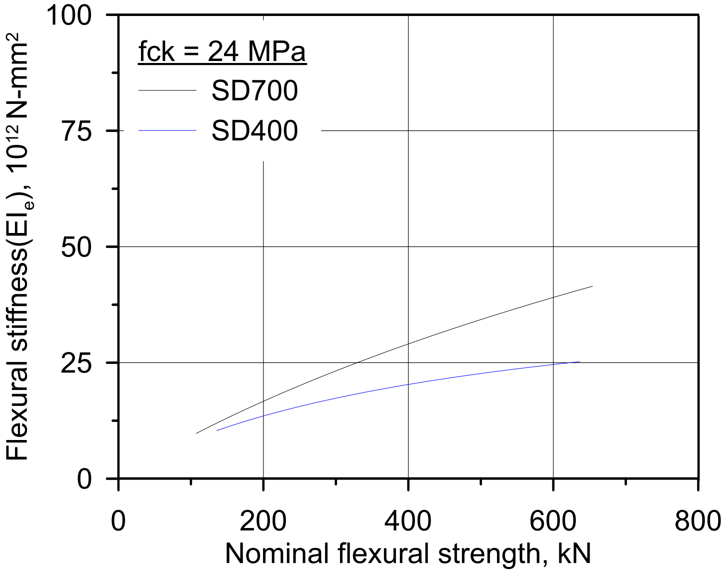


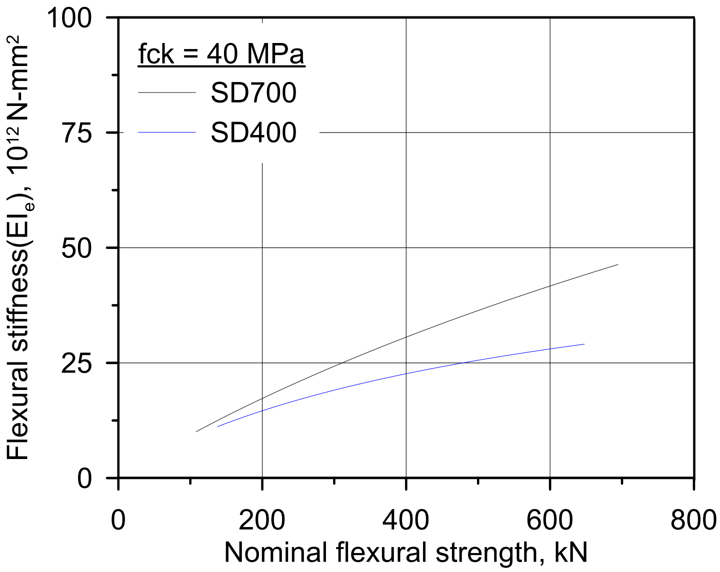
Figure 4.6 Curvature under service load

Comparison of analysis value of members reinforced with SD400 and SD700 rebars are shown in Figures 4.7 and 4.8.



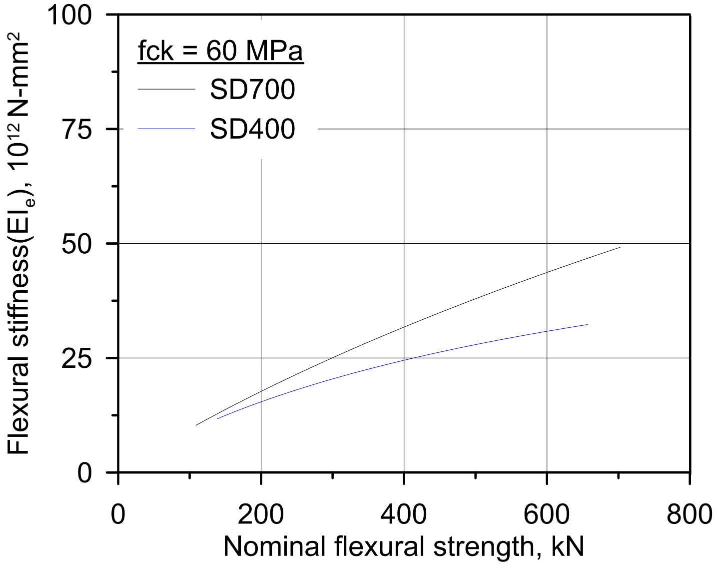
100
75
50
25
0
0
200
400
600
800
fck = 24 MPa
SD700
SD400
Flexural stiffness(EI_e), 10^12 N-mm^2
Nominal flexural strength, kN

(a)



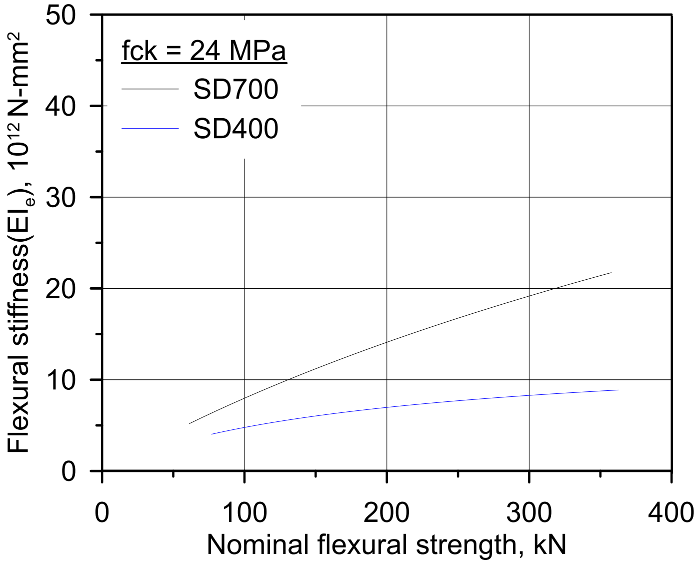
100
75
50
25
0
0
200
400
600
800
fck = 40 MPa
SD700
SD400
Flexural stiffness(EI_e), 10^12 N-mm^2
Nominal flexural strength, kN

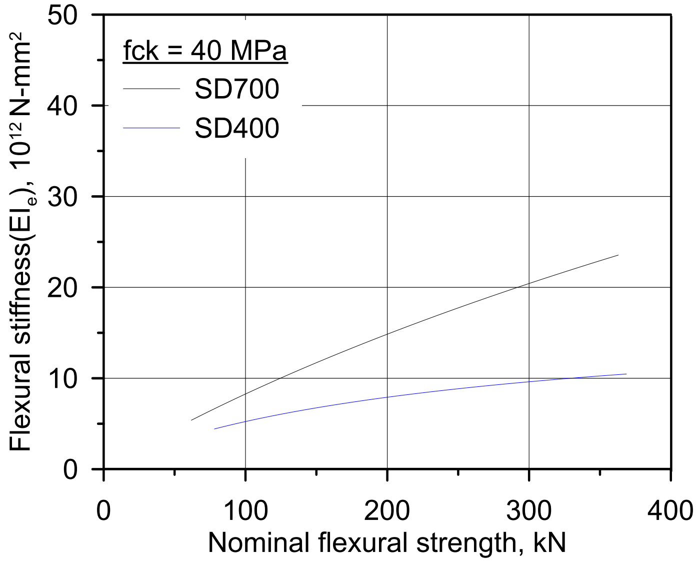
(b)

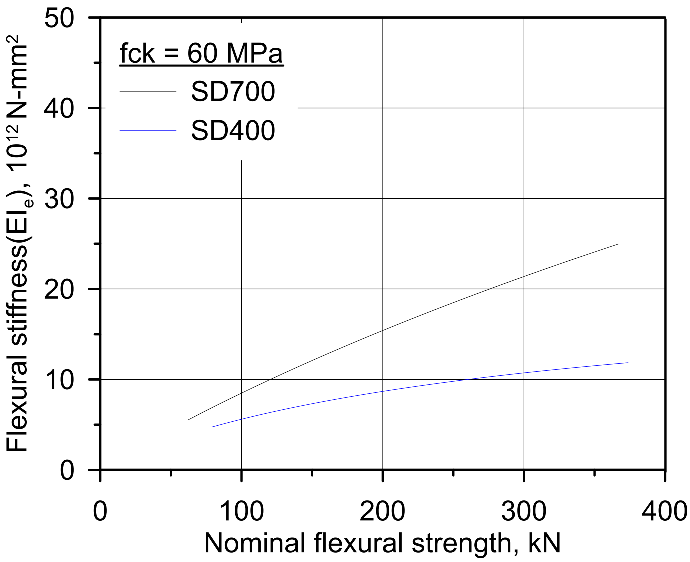


(c)

Figure 4.7 Comparison for effective stiffness of RC beam; (a) $f_{ck} = 24$ MPa, (b) $f_{ck} = 40$ MPa, (c) $f_{ck} = 60$ MPa


fck = 24 MPa
SD700
SD400
Flexural stiffness(EIe), 10^12 N-mm^2
Nominal flexural strength, kN
0
10
20
30
40
50
0
100
200
300
400
(a)


fck = 40 MPa
SD700
SD400
Flexural stiffness(EIe), 10^12 N-mm^2
Nominal flexural strength, kN
0
10
20
30
40
50
0
100
200
300
400
(b)



(c)

Figure 4.8 Comparison for effective stiffness of one way slab; (a) $f_{ck} = 24$ MPa, (b) $f_{ck} = 40$ MPa, (c) $f_{ck} = 60$ MPa

5. Conclusions

This study confirmed that the code provisions related to crack and deflection on SD700 are applicable for Korean existing concrete design code. Experiment and analysis were performed for evaluation of applicability for code provisions on SD700. The following conclusions can be drawn from the work presented in this study.

First, the yield strength of rebar is main variable in most design codes. General regulation is similar in design codes. Crack width is controlled by rebar spacing. Deflection is controlled by the height of structure. Details of code provisions differ in the various design codes. Consequently, research on high strength rebar must be conducted in consideration of the respective design codes. Currently, using SD700 rebar remains limited under the Korean design code. This is due to insufficient research on SD700 rebar in Korea.

Second, average cracking width in service load was increased when using SD700 rebar. Most of average cracking widths were under 0.4 mm, which is the limit value for design provision. Cracks were distributed evenly, with cracks of RC beams and one way slabs focused in the center area and at a reduced number. Thus, when applying current design provisions, cracking width of specimens is deemed acceptable.

Third, for deflection, design provision on minimum height of members was applied to RC beams and one way slabs reinforced with SD700 and SD400 rebars for evaluation. Due to absence of test data on specimens reinforced with SD400 rebar, applicability for code provision related to deflection on SD700 was assessed using analysis. Analysis results indicate that, regardless of concrete compressive strength, the effective stiffness of members reinforced with SD700 is greater than that of SD400. This means deflection of members reinforced with SD700 can be considered compliant with the current design provision on minimum height of members.

In conclusion, this study seeks to fill the need for research on, SD700 rebar, which has currently not been allowed in the Korean design code. Design provisions on indirect crack and deflection control were covered in this study. Using experimental data and analysis, the study results clearly support the applicability of current design provisions on crack and deflection for use of SD700 rebar. Further research is needed on additional code provisions related to flexure and other serviceability requirements in order to fully evaluate the applicability of the design code on SD700 rebar.

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국문초록

균열과 처짐 제어 규정에 대한
SD700 철근 적용성 평가

Tuvd Lkhagvadorj

토목분야에서는 목재, 강철, 아스팔트 및 포틀랜드 시멘트 콘크리트와 같은 전통적인 건축 자재가 많은 건설 프로젝트에서 보통 사용된다. 이러한 재료에 대한 수행된 수많은 연구들은 각 재료들에 대한 이해도를 높였으며, 강도와 내구 성능을 증가시키는 성과를 보였다. 근데에 사용되는 재료들은 과거의 재료보다 더 우수하며, 토목 공학 응용 분야의 특정 요구를 충족시키기 위해 새로운 재료도 개발되고 있다.

건설에 사용되는 콘크리트는 대부분 보강 재료와 조합하여 사용되며, 강철은 가장 흔한 보강 재료로 쓰인다. 일반적으로 철근 콘크리트는 대규모 사회기반시설에 사용된다. 이러한 대규모

사회기반시설 건설에는 적절한 철근의 종류와 양이 중요한 요소이다. 일반 강도 철근을 구조물에 사용하는 경우, 강도 및 사용성에 대한 요구 사항을 충족하기 위해 큰많은 양의 철근을 필요로 한다. 그러나 과도한 철근은 콘크리트 품질에 영향을 미칠 수 있다. 이 문제를 방지하는 한 가지 적절한 방법은 다양한 이점을 제공하는 고강도 철근을 적용하는 것이다.

반면에, 철근 콘크리트 구조물에 고강도 철근의 사용은 다른 문제들을 야기할 수 있다. 고강도 철근을 사용할 시 철근의 혼잡이 감소하는 반면에 철근 사이의 간격이 증가하여 균열폭과 처짐을 초래할 수 있다. 따라서 고강도 철근을 사용하는 경우 철근 콘크리트의 사용성에 대한 사항은 필수적이다. 이 점에 비추어 철근의 최대 항복 강도는 콘크리트구조 설계기준에서 제한된다. 한국 콘크리트구조 설계기준(KDS)에서는 SD600 철근이 이미 허용되었다. 고강도 SD700 철근에 대한 연구도 진행되었으나 사용성에 대한 추가 연구가 필요하여 아직 국내 콘크리트구조 설계기준에 포함되지 않았다.

이 연구는 앞서 언급된 고강도 철근의 사용에 대한 문제점을 해결하고자 한다. 본 연구에서는 균열폭과 처짐 제어 규정에 대한 SD700 사적용성을 평가하기 위해 SD700 철근이 배근된 철근 콘크리트 보와 1 방향 슬래브에 대한 휨 실험을 진행하였다. 마지막으로 사용성 평가를 위해 KDS 에 제시된 부재의 허용균열폭과 최소 높이에 대한 규정을 제시하였다.

주요어: 고강도 철근, 철근 항복강도, 허용균열폭, 부재의 최소 높이, 유효강성

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