1st Draft

# Design Guidelines for K-UHPC

2014. 2.

KICT 한국건설기술연구원

# Foreword

Active research is conducted worldwide on ultra high performance concrete (UHPC) to improve the properties and complement the disadvantages of concrete, as the most widely adopted construction material. Especially, the research on UHPC led by the Korea Institute of Construction Technology (KICT) in Korea has completed the development of original design technology after having passed through the material development and validation stage and, has realized its application in the design of real bridge structures. However, the application and expansion of this new construction material had been impeded by the delayed publication of authorized design guidelines that would supply bases and specifications to the designers. As an effort to overcome this situation, the PSC advisory committee of the Korea Concrete Institute established the "Design Guidelines for Ultra High Performance Concrete (K-UHPC) Structure".

These guidelines, based upon the structure and contents of the "Structural Concrete Design Code" (2012) enacted by the Korea Concrete Institute, reflect the latest worldwide research results and the achievements of『Super Bridge 200』Project of KICT and, compile and analyze foreign specifications dedicated to the design of UHPC structures under the supervision of experts and designers who participated to the establishment of the "Structural Concrete Design Code".

Considering the specificity of our technology related to UHPC, these guidelines were prepared by focusing on the design of structures using K-UHPC, the UHPC developed by KICT. This implies that the application of the contents of these guidelines for other UHPC shall be done after sufficient understanding of the proposed recommendations and verification of the performances through appropriate tests. Especially, the mechanical properties of the material may vary according to the fabrication method and quality control of UHPC. Therefore, the performances shall be sufficiently verified according to the standardized testing method proposed in the Appendices and appropriate reflection of the corresponding results in the design.

To conclude, I hope that these guidelines will contribute to the application and expansion of UHPC in Korea and be of valuable help to the designers. In addition, I express my gratitude to the contributors who shared time and tremendous energy in preparing these guidelines.

2014. 2.

Executive authors **Kim, Jee-Sang**

**Kim, Byung-Suk**

# Contents

Chapter 1

# General Provisions

## Specifications

### 1.1 Objectives

These provisional guidelines are to specify the minimal requirements necessary to secure the safety, serviceability and durability of structures using the Ultra High Performance Concrete developed by the Korea Institute of Construction Technology (hereinafter referred to as K-UHPC).

### 1.2 Scope of Application

(1) These guidelines specify the general and basic requirements necessary for the design of K-UHPC structures.

(2) The design of K-UHPC structures apply the strength design method presented in these guidelines.

(3) Items not specified in these guidelines shall follow the relevant design code or design guidelines enacted by the Ministry of Land, Transportation and Maritime Affairs.

(4) The results applying the specifications of these guidelines prevail when the results applying existing design standards related to

## Commentary

### 1.1 Objectives

These provisional guidelines specify the technical features needed to design structures using K-UHPC. Features not specified in these guidelines shall follow the regulations prescribed in “Structural Concrete Design Code” and “Standard Specifications for Concrete”.

### 1.2 Scope of Application

(1) These guidelines do not regulate all the items that may be encountered in the design but provide specifications only for the most basic factors or items. Accordingly, items that are not specified in these guidelines shall be designed with respect to currently applicable theories, test results and experience at the discretion of the technician in charge.

(4) Even if these guidelines compile the latest research achievements related to K-UHPC, all the regulations specified in existing relevant

**Specifications**

items not specified in these guidelines enter in conflict with the specifications of these guidelines.

**Commentary**

design standards are not examined. Accordingly, in case of application of other specifications required for the design but absent in these guidelines, the corresponding results may disagree with those obtained by applying the regulations of these guidelines. In such case, the results applying the regulations of these guidelines prevail considering the specific characteristics of K-UHPC, which differ from ordinary concrete.

Chapter 2

# Material Properties

## Specifications

### 2.1 General

(1) This chapter presents general provisions necessary for design in concern with the material properties of K-UHPC. Refer to "Structural Concrete Design Code" for the material properties of steel reinforcements and prestressing tendons (PS tendons).

(2) The material properties of K-UHPC can be expressed in terms of the compressive strength, tensile strength, other strength characteristics, elastic modulus, shrinkage and creep, thermal characteristics, durability, watertightness according to necessity. The effect of the reinforcing fiber shall be taken into account for the strength characteristics and deformation characteristics. The effect of the loading speed shall also be considered if necessary.

(3) The characteristic strength, $f_k$, of K-UHPC, which considers the variation of the experimental data, is set to a value so that the experimental data are smaller than this value with a probability below 5%.

(4) The design strength, $f_d$, of K-UHPC is the characteristic strength, $f_k$, multiplied by the material reduction factor, $\phi_m$.

## Commentary

### 2.1 General

(1) Since the steel reinforcement and tendon are identical to those used in conventional concrete structures and there is no difference in their material properties, design shall apply the design values of "Structural Concrete Design Code".

(2) K-UHPC exhibits different material properties according to various conditions including the adopted materials and curing process. Moreover, the shape, mixing ratio and utilization method of the reinforcing fiber influence significantly the compressive strength, tensile strength and behavioral characteristics in the softening zone. Therefore, K-UHPC used in structures or members shall be selected so as to satisfy appropriate performance considering the purpose of use, environmental conditions and construction conditions.

(3) In cases where the effect of the strain rate must be considered as in impact loads, values obtained from highly reliable tests shall be adopted.

(4) Here, the design strength, $f_d$, and the characteristic strength, $f_k$, designate comprehensively the compressive strength,

**Specifications**

(5) The material reduction factor, $\phi_m$, of the steel reinforcement and tendon is set to a value of 1.0.

2.1.1 Nomenclature

$A_c =$ cross-sectional area of member (mm$^2$)
$E_{ct} =$ elastic modulus of K-UHPC under loading (MPa)
$f_{cd} =$ design compressive strength of K-UHPC (MPa)
$f_{ck} =$ characteristic value of compressive strength of K-UHPC (MPa)
$f_{cm} =$ average compressive strength of K-UHPC used when calculating creep (MPa)
$f_{cm0} =$ constant used to calculate creep (10 MPa)
$f_{crd} =$ design crack strength of K-UHPC (MPa)
$f_{crk} =$ characteristic crack strength of K-UHPC (MPa)
$f_d =$ design strength of K-UHPC (MPa)
$f_k =$ characteristic strength of K-UHPC (MPa)
$f_{rd} =$ design fatigue strength of K-UHPC (MPa)
$f_{td} =$ design tensile strength of K-UHPC (MPa)
$E_c =$ elastic modulus of K-UHPC (MPa)
$f_{tk} =$ characteristic tensile strength of K-UHPC (MPa)
$G_F =$ fracture energy (N/mm)
$h =$ conceptual member dimension (mm)
$h_{beam} =$ height of beam (mm)
$L_{eq} =$ equivalent length used to transform the tension softening curve into tensile stress-strain curve considering the

**Commentary**

tensile strength, crack strength, bond strength and, bearing strength. In addition, the material reduction factor, $\phi_m$, stands comprehensively for all the materials used in the K-UHPC structure including K-UHPC, steel reinforcement, and tendon. Besides, the material reduction factor dedicated to K-UHPC only is designated as $\phi_c$.

2.1.1 Nomenclature

**Specifications**

height of the beam (mm)

$l_{ch}=$ characteristic length ( $= G_F E_c / f_{tk}^2$ ) (mm)

$N=$ fatigue life

$t=$ age of K-UHPC (day)

$t_1=$ term introduced to express time function in dimensionless form, 1-day age

$t_s=$ age of concrete at initiation of drying shrinkage and expansion (day)

$u=$ circumferential length of cross-sectional area $A_c$ (mm)

$W/B=$ water-to-binder ratio

$w_u=$ crack opening displacement or crack width at tensile strength caused by strain softening after cracking (mm)

$w_{\lim}=$ crack opening displacement or crack width once tensile stress vanishes (mm)

$\alpha_{as}=$ coefficient depending on the characteristics of K-UHPC

$\beta_{as}=$ autogenous shrinkage characteristics

$\beta_c=$ function of age

$\beta_{ds}=$ coefficient expressing the development of autogenous shrinkage according to age that can be used in the design $(2A_c/u)$

$\beta_s=$ coefficient expressing the development of drying shrinkage with time

$\phi_c=$ material reduction factor of K-UHPC

$\phi_m=$ material reduction factor for all the materials used in K-UHPC structure

$\epsilon_{as}=$ autogenous shrinkage at time $t$

$\epsilon_{as0}=$ conceptual autogenous shrinkage coefficient

$\epsilon_{cc}=$ creep strain of K-UHPC

$\epsilon_{cr}=$ strain at initiation of crack

$\epsilon_{ds}=$ total drying shrinkage strain at age $t$ of K-UHPC exposed to open air at $t_s$

$\epsilon_{ds0}=$ conceptual drying shrinkage coefficient

$\epsilon_{st}=$ strain at tensile strength

$\epsilon_{sh}=$ drying shrinkage strain

**Commentary**

**Specifications**

$\epsilon_u =$ ultimate strain at tension
$\sigma_{cp} =$ developed stress
$\sigma_p =$ stress caused by permanent load
$\phi =$ creep coefficient
$\phi_0 =$ conceptual creep coefficient

## 2.2 Basic Mix

The basic mix of K-UHPC shall secure a characteristic compressive strength higher than 180 MPa but can exhibit different mix composition and constitutive materials of K-UHPC according to the performances required by the structure.

**Commentary**

## 2.2 Basic Mix

Table c.2.1.1 presents an example of the basic mix composition of K-UHPC. The W/B is 0.2, and the powder combines cement, silica fume, filler and shrinkage reducing agent. The mix uses fine aggregates as aggregates. Liquid superplasticizer is adopted to secure the fluidity with an amount varying with respect to the required fluidity and environmental conditions. Steel fiber is admixed at a volume fraction of 1.5~2% relative to the whole volume according to the required tensile strength and type of steel fiber.

**Table c.2.1.1** Example of basic mix composition of K-UHPC (in mass ratio)

| Component | W/B | Cement | Silica fume | Filler | Fine aggregates | Shrinkage reducing agent | Superplasticizer | Steel fiber (volume fraction) |
|---|---|---|---|---|---|---|---|---|
| Mix ratio | 0.2 | 1 | 0.25 | 0.3 | 1.1 | 0.085 | 0.018 | 1.5~2% of volume of K-UHPC |

The properties of the materials used in K-UHPC are as follows.

① Cement and reactive powder: Ordinary Portland cement is adopted as cement and silica fume is used as reactive powder. Silica fume shall exhibit specific surface area larger than 150,000 ㎠/g and contain more than 96% of $SiO_2$.

② Aggregates: The aggregates used in K-UHPC shall be fine aggregates made of quartz sand with diameter smaller than 0.5 mm and $SiO_2$ content larger than 96%.

**Specifications** | **Commentary**

Coarse aggregates are not used.

③ Filler: The filler shall use a material with average particle size of 4㎛ and $SiO_2$ content larger than 96%.

④ Superplasticizer: Polycarboxylate superplasticizer (density 1.01 g/㎤, dark brown liquid, solid content 30%) shall be used.

⑤ Shrinkage reducing agent: The shrinkage reducing agent shall be a combination of CSA type expansion admixture and glycol-based shrinkage reducing agent. The recommended mixture ratio is basically 7.5% of expansion admixture and 1% of shrinkage reducing agent but can vary according to the amount of shrinkage allowed by K-UHPC.

⑥ Steel fiber: Steel fiber shall basically exhibit tensile strength higher than 2,000 MPa and diameter of 0.2 mm and the length shall be selected appropriately among the values of 13 mm, 16 mm and, 20mm according to the tensile strength required for K-UHPC.

## 2.3 Curing

(1) The curing process of K-UHPC is in principle conducted first by the early curing after placing followed by high temperature wet curing.

(2) The temperature and duration of the high temperature wet curing shall be decided considering the material characteristic values of K-UHPC necessary for design according to its place of use and purpose.

## 2.3 Curing

(1), (2) The early curing after placing shall maintain the temperature and humidity necessary for hardening until securing the required stripping strength and prevent harmful action. Early curing shall be conducted during 12 to 48 hours under wet condition at 20±2℃ according to circumstances but a basic duration of 24 hours is recommended. K-UHPC that is wet cured at high temperature is expected to exhibit fast strength development, reduced shrinkage and creep, and improved durability owing to the densification of its structure. High temperature wet curing shall be performed at 90±5℃ during 24 to 72 hours according to the conditions of the structure

**Specifications** | **Commentary**

but a basic duration of 48 hours is recommended. In addition, in case where high temperature wet curing must be conducted at temperature different to 90±5℃ due to the limitation of the curing equipment, the satisfaction of the performances of K-UHPC shall be verified in advance.

## 2.4 Strength

### 2.4.1 Characteristic Strength

(1) The characteristic strength of K-UHPC shall be determined in principle after completion of high temperature wet curing in accordance to the test specified in the appendix of these guidelines.

## 2.4 Strength

### 2.4.1 Characteristic Strength

(1) The strength development characteristics of K-UHPC depends sensitively on the curing method. Since the execution of high temperature wet curing promotes significantly the development of strength, stable strength is achieved after completion of high temperature wet curing and the effect of age on the strength reduces. For K-UHPC fabricated using the basic mix composition and wet cured at high temperature, the compressive strength becomes nearly constant regardless of the age as shown in Figure c.2.4.1. Accordingly, it is recommended to determine the characteristic strength by test after completion of the high temperature wet curing.

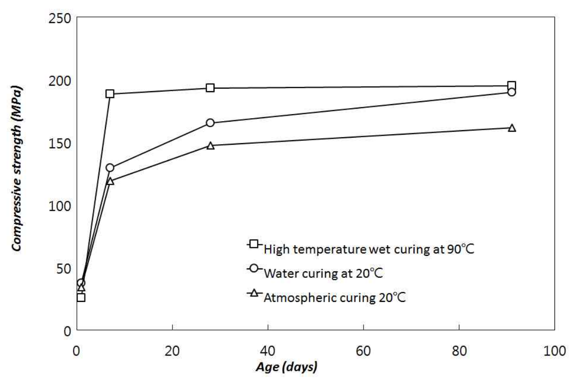



Fig. c.2.4.1 Effect of curing on the compressive strength

(2) The design strength of K-UHPC not subjected to high temperature wet curing shall be determined by test at the appropriate age according to the purpose of use, the

(2) When high temperature wet curing is not performed, the strength increases with the age similarly to ordinary concrete if K-UHPC is cured appropriately. Following, the strength

**Specifications**

loading time of major loads, construction plan of the structure.

(3) The value of the design tensile strength shall be obtained by multiplying the characteristic tensile strength by the material reduction factor including the effect of the orientation of the steel fibers.

**Commentary**

shall be determined by test at the appropriate age according to the purpose of use, the loading time of major loads, construction plan of the structure.

(3) Difference may occur due to the orientation of the steel fibers between the strength of K-UHPC obtained through specimens and the strength of K-UHPC of the actual structure. For example, the French UHPFRC Interim Recommendations of AFGC-SETRA (2002, hereinafter referred to as AFGC-SETRA Recommendations), dedicated to a similar type of UHPC, introduces the fiber orientation coefficient $K_f$ (ratio of the tensile strength of the specimen for quality control to that of the specimen extracted from actually erected structure) to consider the difference in the orientation of the fibers in actually constructed structures and in specimens fabricated for quality control. The AFGC-SETRA Recommendations specify to divide the tensile strength of the specimen by $K_f$ so as to account for the effect of the orientation of the fibers in the design. The AFGC-SETRA Recommendations recommend to determine the value of $K_f$ through full scale or reduced scale test during the construction planning stage or to use $K_f = 1.25$ (in case of usual axial force, flexure and shear) or $K_f =$ 1.75 (in case of local loading) based on previous test results when test is not performed.

Besides, the Japanese Provisional Recommendations for the Design and Construction of Ultra High Strength Fiber-Reinforced Concrete of JSCE (2004, hereinafter referred to as JSCE Recommendations) consider the change in the material properties including the orientation of the steel fibers by the material factor $\gamma_c = 1.3$.

Since these guidelines also apply a design concept considering a material reduction

**Specifications**

2.4.2 Compressive Strength

The characteristic compressive strength, $f_{ck}$, of K-UHPC shall be determined based upon the compressive strength obtained by means of appropriate test.

2.4.3 Crack Strength

The characteristic crack strength, $f_{crk}$, of K-UHPC shall be determined based upon the crack strength obtained through appropriate

**Commentary**

factor, the effect of the orientation of the steel fibers is reflected by including it in the material reduction factor.

2.4.2 Compressive Strength

The compressive strength of K-UHPC shall conform with the "Compressive Strength Test Method of K-UHPC" provided in Appendix 2 of these guidelines.

Figure c.2.4.2 presents the frequency distribution of the compressive strength resulting from tests on cylinder specimens with diameter of 100 mm made of K-UHPC fabricated using the basic mix composition and wet cured at high temperature. The average compressive strength of K-UHPC is 198.68 MPa with standard deviation of 9.49 MPa. Assuming a normal distribution, the compressive strength is found to be 188.75 MPa if a probability of 5% is calculated for the compressive strength of the cylinders to fall below the characteristic strength. Accordingly, a conservative value of 180 MPa can be recommended for the characteristic compressive strength, $f_{ck}$.

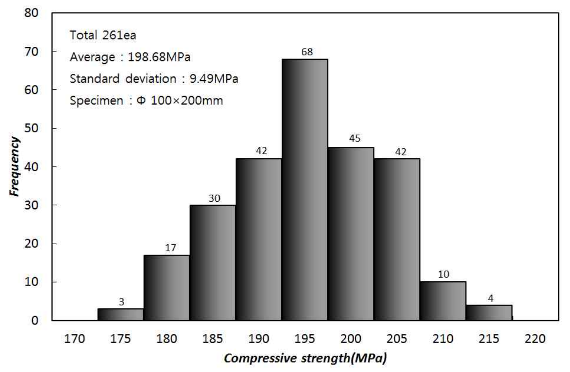



Fig. c.2.4.2 Frequency distribution of compressive strength resulting from strength test on 100 mm−diameter cylinders

2.4.3 Crack Strength

K-UHPC subjected to a tensile force starts to show linear elastic behavior, which vanishes with the initiation of cracking as the load

## Specifications

test.

2.4.4 Tensile Strength and Tension-Softening Characteristics

(1) The characteristic tensile strength, $f_{tk}$, and tension-softening characteristics of K-UHPC shall be determined based upon appropriate test.

## Commentary

increases. The crack strength designates the stress in the tensile stress-strain curve at which the linear elastic behavior vanishes following the initiation of crack. The crack strength can be obtained through direct tensile test or inverse analysis of bending test results. However, these guidelines specify to compute the crack strength by means of the "Direct Tensile Test Method for K-UHPC (unnotched specimen)" of Appendix 4 or the "Direct Tensile Test Method for K-UHPC (notched specimen)" of Appendix 5.

Table c.2.4.1 arranges the crack strength obtained from the results of direct tensile test performed on K-UHPC fabricated using the basic mix composition (1% of 16 mm-long steel fiber with diameter of 0.2 mm and 1% of 20 mm-long steel fiber with diameter of 0.2 mm) and wet cured at high temperature. It can be seen that a characteristic crack strength of 9.5 MPa can be used conservatively until report of new results.

**Table c.2.4.1** Crack strength of K−UHPC

| | Crack strength (MPa) | | | |
|---|---|---|---|---|
| | Average | Std. deviation | Design characteristic value | Recommended characteristic value for design |
| Direct tensile test | 12.18 | 1.56 | 9.61 | 9.5 |

2.4.4 Tensile Strength and Tension-Softening Characteristics

(1) The tension-softening curve

The tension-softening curve represents the relation between the tensile stress and the crack width and shall be obtained using the "Direct Tensile Test Method for K-UHPC (notched specimen)" of Appendix 5. Unlike ordinary concrete, K-UHPC experiences gradual increase of the transferred tensile stress owing to the steel fibers according to the enlargement of the crack width after the

## Specifications

(2) The shape of the tension-softening curve can also be represented by the model shown in Figure 2.4.1.

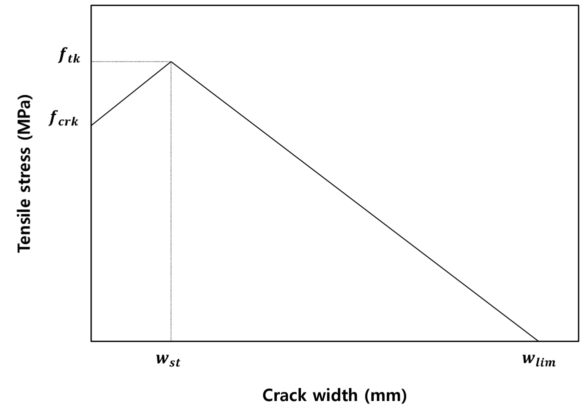


Fig. 2.4.1 Tension−softening curve

In Figure 2.4.1, $w_u$ designates the crack width at tensile strength caused by strain-hardening after the initiation of crack and, $w_{\lim}$stands for the crack width at which tensile stress vanishes.

## Commentary

initiation of crack to reach its maximum value with the tensile strength, followed by gradual decrease which ends finally to the formation of cracks that do not transfer the load.

The methods to obtain the tensile strength and tensile stress-deformation (crack width) characteristics of K-UHPC are the direct tensile test and bending tensile test. The bending tensile test is an indirect method providing the tensile stress-strain relationship by inverse analysis of the experimental results, whereas the direct tensile test is a method obtaining directly the tensile stress-strain relation similarly to the tensile test of steel. These guidelines recommend the direct tensile test as basic test method enabling to obtain directly the tensile stress-strain relationship from standard specimens rather than the indirect method, of which results vary according to the numerical analysis model and inverse analysis method.

(2) The tensile behavior of K-UHPC shall consider the strain-hardening. To that goal, the shape of the tension-softening curve in Figure 2.4.1 can be defined by test. In other words, the characteristic crack strength, $f_{crk}$, and characteristic tensile strength, $f_{tk}$, can be derived from the crack strength and tensile strength obtain by direct tensile test of K-UHPC.

In the absence of test, the value recommended for design in Table c.2.4.1 can be used for the characteristic crack strength, $f_{crk}$, and the value recommended for design in Table c.2.4.2 can be adopted for the characteristic tensile strength, $f_{tk}$. Table c.2.4.2 arranges the tensile strength obtained from the results of direct tensile test performed on K-UHPC fabricated using the basic mix composition (1% of 16 mm-long steel fiber with diameter of 0.2 mm and 1% of 20

## Specifications

## Commentary

mm-long steel fiber with diameter of 0.2 mm) and wet cured at high temperature.

Here, the crack width, $w_u$, at the instant at which the tensile strength is developed can be set as 0.3 mm. Referring to the experimental results (Figure c.2.4.3). this value is conservative owing to the absence of case reaching the tensile strength with crack width shorter than 0.3 mm. In addition, the AFGC-SETRA Recommendations also define the tensile strength at 0.3 mm as the limit value of the tensile strength.

The crack width, $w_{\lim}$, at the state where tensile stresses do not exist anymore, can be set to 5.3 mm considering the test results (Figure c.2.4.3).

Accordingly, the following values can be used in the case of K-UHPC fabricated using the basic mix composition and wet cured at high temperature: characteristic crack strength $f_{crk}$ = 9.5 MPa, characteristic tensile strength $f_{tk}$ = 13.0 MPa, $w_u$ = 0.3 mm and, $w_{\lim}$ = 5.3 mm.

Table c.2.4.2 Tensile strength of K-UHPC

| | Tensile strength (MPa) | | | |
|---|---|---|---|---|
| | Average | Std. deviation | Design characteristic value | Recommended characteristic value for design |
| Direct tensile test | 15.33 | 1.38 | 13.07 | 13.00 |

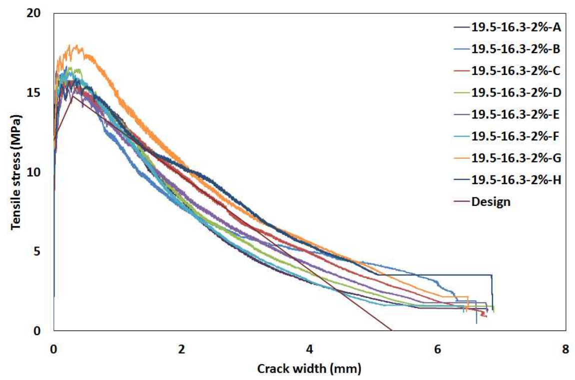



Fig. c.2.4.3 Characteristic tensile stress-crack width curves obtained by direct tensile test

**Specifications**

2.4.5 Other Strengths

The characteristic bond strength and bearing strength of K-UHPC shall be determined based upon the bond strength and bearing strength obtained by appropriate tests.

**Commentary**

2.4.5 Other Strengths

Owing to the high bond strength of K-UHPC, the length of the tendons for prestressing and the anchored length of the stiffening steel in the connection can be shortened compared to those applied in ordinary concrete. In addition, the high bearing strength enables also to reduce the dimensions of the tendon anchorage for prestressing compared to those in ordinary concrete.

However, due to the current lack of experimental data related to K-UHPC, the characteristic bond strength and bearing strength of K-UHPC shall be determined based upon the bond strength and bearing strength obtained by appropriate tests.

In the absence of test, the bond strength and bearing strength may comply with the regulations of the “Structural Concrete Design Code”.

In view of the test results of the Korea Institute of Construction Technology, a minimum bond length of about 3 times the diameter appeared to secure sufficient bond behavior as indicated in the rebar pull out test of RILEM for D10, D13, D16, D19 and D22 deformed reinforcements. However, only a small portion of the stiffening member is embedded in K-UHPC and the length of the embedded stiffening member necessary for the ultimate bond load to reach the yield load of the rebar was found to be at least 6 times the diameter through pull out test by applying direct loading.

## 2.5 Stress-Strain Curves

**Specifications**

2.5.1 Compressive Stress-Strain Curve

(1) The compressive stress-strain curve of K-UHPC shall be determined by test or shall apply the idealized compressive stress-strain curve shown in Figure 2.5.1.

## 2.5 Stress-Strain Curves

**Commentary**

2.5.1 Compressive Stress-Strain Curve

(1) The compressive strength of K-UHPC is based upon “Appendix 2: Compressive Strength Test Method for K-UHPC”. For K-UHPC, the deformation performance can be

## Specifications

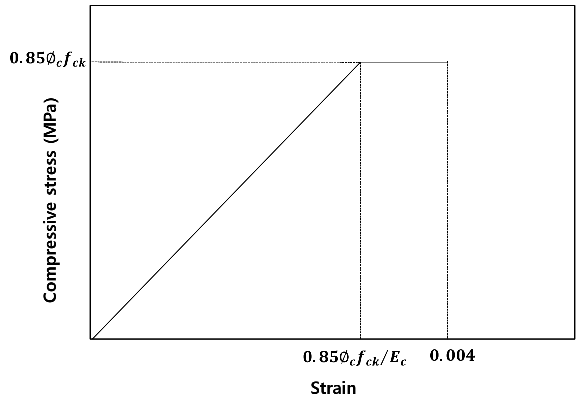



Fig. 2.5.1 Compressive stress−strain curve of K−UHPC

(2) The idealized compressive stress-strain curve presented in Figure 2.5.1 can generally be used for the examination of the section forces at the ultimate state of a member subjected to flexural moment or simultaneously to flexural moment and axial compressive force. In such case, the material reduction factor $\phi_c$ is set to 0.91 and the elastic modulus $E_c$ is determined with respect to Section 2.6. In addition, the design compressive strength $f_{cd}$ becomes $\phi_c f_{ck}$.

## Commentary

improved after the compressive strength owing to the admixing of steel fiber. Figure c.2.5.1 plots the compressive stress-strain behavior of K-UHPC measured on cylinders with diameter of 100 mm.

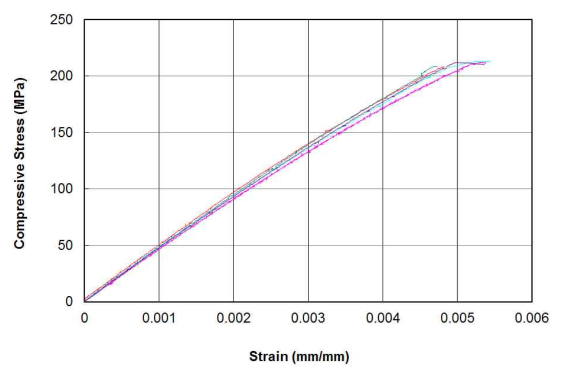



Fig. c.2.5.1 Compressive stress−strain curves

(2) Since the post-compressive strength's deformation performance is affected by the effects of the type, shape and mix ratio of the reinforcing fiber, it is necessary to set appropriately the post-compressive strength's stress-strain curve based upon the actual behavior when examining the deformation and toughness at ultimate state of beam members. The curve shown in Figure 2.5.1 can be used in the examination of the section forces at ultimate state. Here, a rating factor of 0.85 shall be applied to the characteristic compressive strength considering the difference between the strength calculated from the internal forces of the member (reduction of the compressive strength under long-term loading) and the strength of the specimen.

Unlike the JSCE Recommendations which define a material factor for common UHPC, these guidelines relate a specific type UHPC that is K-UHPC. Therefore, these guidelines specify a higher material reduction factor, $\phi_c$, of 0.91 for the compressive strength of K-UHPC of which variability is relatively well evaluated based upon sufficient tests.

## Specifications

(3) In biaxial and triaxial stress state, the adopted compressive stress-strain curve of K-UHPC shall consider the effect of multi-axial stress state.

### 2.5.2 Tensile Stress-Strain Curve

(1) Tension-softening curve or tensile stress-strain curve with appropriate shape shall be assumed for the examination of the strength of K-UHPC members.

(2) The modelized tensile stress-strain curve plotted in Figure 2.5.2 can generally be used for the examination of the strength at ultimate state of members subjected to flexural moment or flexural moment and axial compressive force. in such case, $L_{eq}$ shall be obtained using Equation (2.5.1) and the material reduction factor shall be 0.8.

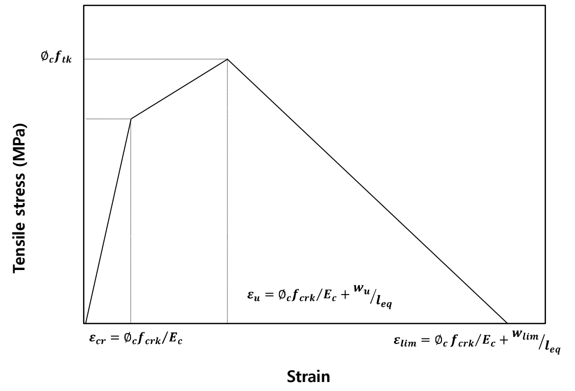



Fig. 2.5.2 Tensile stress−strain curve of K−UHPC

$$L_{eq}/h_{beam} = 0.8\left[1 - 1/(1.05 + 6h_{beam}/l_{ch})^4\right] \quad (2.5.1)$$

where

$h_{beam}$: height of beam (mm)

$l_{ch}$ : characteristic length (= $G_F E_c / f_{tk}^2$ = $1.01\times10^4$ mm)

$G_F$ : fracture energy (= 37.9 N/mm)

$E_c$ : elastic modulus (= $4.5\times10^4$ MPa)

$f_{tk}$ : tensile strength (= 13.0 MPa)

## Commentary

(3) The compressive stress-strain curve of K-UHPC described in Figure 2.5.1 is based upon uniaxial stress state. Since behavior different from that of Figure 2.5.1 will occur for biaxial and triaxial stress state, such effect shall be considered if necessary.

### 2.5.2 Tensile Stress-Strain Curve

(1) Even if the tensile stress-strain curve of K-UHPC is proposed in "Appendix 4: Direct Tensile Strength Test Method for K-UHPC (unnotched specimen)", the methodology presented here shall be applied with regard to the behavioral characteristics of K-UHPC.

(2) In case where the tension-softening curve is not obtained by test, the stress-strain curve shown in Figure 2.5.2 can be applied for the examination of the strength. This curve by converting the tension-softening curve of Figure 2.4.1 into tensile stress-strain curve using the equivalent length $L_{eq}$ of the JSCE Recommendations.

Similarly to the compressive strength, these guidelines relate a specific type UHPC that is K-UHPC unlike the JSCE Recommendations which define a material factor for common UHPC. Therefore, these guidelines specify a higher material reduction factor, $\phi_c$, of 0.8 for the tensile behavior of K-UHPC of which variability is relatively well evaluated based upon sufficient tests.

**Specifications**

$f_{td}$ : design tensile strength ($f_{td} = \phi_c f_{tk}$ = 10.4 MPa)
$f_{crk}$ : characteristic crack strength (= 9.5 MPa)
$f_{crd}$ : design crack strength ($f_{crd} = \phi_c f_{crk}$ = 7.6 MPa)

## 2.6 Elastic Modulus

(1) The elastic modulus of K-UHPC shall be determined based upon appropriate test.

(2) In the absence of specimen, a value of $4.5 \times 10^4$ MPa shall be used for the characteristic elastic modulus of K-UHPC fabricated using the basic mix composition and wet cured at high temperature.

## 2.7 Poisson's Ratio

The Poisson's ratio of K-UHPC is set to 0.2 in the elastic range.

## 2.8 Thermal Properties

The thermal properties of K-UHPC shall be determined based upon test or previous data.

**Commentary**

## 2.6 Elastic Modulus

(1) The elastic modulus of K-UHPC shall be obtained in compliance with KS F 2438 "Testing Method for Static Modulus of Elasticity in Compression of Cylindrical Concrete Specimens". However, considering the high compressive strength characteristics of K-UHPC, the elastic modulus can also be obtained using the strength and strain corresponding to 10% and 30% of the compressive strength as proposed by FHWA (2006).

(2) Based upon experimental data gathered previously, the characteristic elastic modulus of K-UHPC fabricated using the basic mix composition and wet cured at high temperature is $4.5 \times 10^4$ MPa. Moreover, the elastic moduli in compression and tension within the elastic range can be verified to be practically equal.

## 2.7 Poisson's Ratio

Until report of new experimental results, the Poisson's ratio of K-UHPC can be set to 0.2 referring to the regulations of JSCE Recommendations and AFGC -SETRA Recommendations.

## 2.8 Thermal Properties

Since the thermal properties of K-UHPC exhibit generally extremely large variability according to its water content and the

**Specifications** | **Commentary**

temperature, the thermal properties shall be determined in principle based upon tests.

According to the JSCE Recommendations, UHPC fabricated using the basic mix composition and wet cured at high temperature exhibits the thermal properties arranged in Table c.2.8.1 in air-dry condition at 20℃ after curing.

Until report of new experimental results, the values of Table c.2.8.1 can be used as thermal properties of K-UHPC.

Table c.2.8.1 Thermal properties of K−UHPC after high temperature wet curing

| | |
|---|---|
| Thermal expansion coefficient ($\times 10^{-6}$/℃) | 13.5 |
| Thermal conduction rate (kJ/m·h·℃) | 8.3 |
| Thermal diffusivity ($\times 10^{-3}$㎡/h) | 3.53 |
| Specific heat (kJ/kg·℃) | 0.92 |

## 2.9 Shrinkage

(1) The shrinkage of K-UHPC shall be determined considering the effects of the material properties, mix composition, curing condition, temperature and humidity around the structure and, the shape and dimensions of the member cross-section.

## 2.9 Shrinkage

(1) The shrinkage of K-UHPC is characterized by the large occurrence of autogenous shrinkage and is affected by the effects of various factors including the material properties and curing condition. Following, the shrinkage strain of K-UHPC used in design shall be determined under consideration of these factors.

The total shrinkage including the autogenous shrinkage and drying shrinkage of K-UHPC that has been subjected to high temperature wet curing ranges around $550\times10^{-6}$ as shown in Figure c.2.9.1. Moreover, the total shrinkage of K-UHPC that has been subjected to standard curing ranges around $600\times10^{-6}$.

## Specifications

(2) Equations (2.9.1) and (2.9.2) can be applied to predict the drying shrinkage behavior of K-UHPC after high temperature wet curing.

$$\epsilon_{ds}(t,t_s) = \epsilon_{ds0}\beta_s(t-t_s) \qquad (2.9.1)$$

$$\beta_s(t-t_s) = \left[\frac{(t-t_s)/t_1}{350(h/h_0)^2+(t-t_s)/t_1}\right]^{0.2}$$

or

$$\beta_s(t-t_s) = \left[\frac{(t-t_s)/t_1}{87.5(h/h_0)^2+(t-t_s)/t_1}\right]^{0.5} \qquad (2.9.2)$$

where $\epsilon_{dso}$ stands for the conceptual drying shrinkage; $\beta_s$ is a coefficient representing the development of drying shrinkage with time; $t$ is the age of concrete (day); and, $t_s$ designates the age (day) of concrete at which drying shrinkage and expansion start.

## Commentary

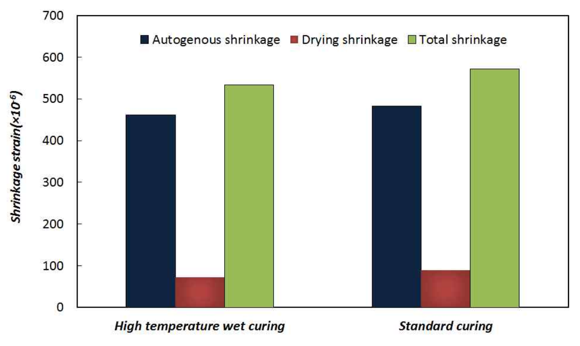



Fig. c.2.9.1 Shrinkage strain of K−UHPC

For information, according to the JSCE Recommendations, the shrinkage of UHPC subjected to high temperature wet curing ranges around $450\times10^{-6}$ at high temperature and ranges around $50\times10^{-6}$ after completion of high temperature curing. In the absence of high temperature curing, the shrinkage strain varies with the age similarly to ordinary concrete and the total shrinkage runs around $550\times10^{-6}$.

(2) $\beta_s$ for the drying shrinkage specified in the "Structural Concrete Design Code" is defined by Equation (c.2.9.1).

$$\beta_s(t-t_s) = \left[\frac{(t-t_s)/t_1}{350(h/h_0)^2+(t-t_s)/t_1}\right]^{0.5}$$

(c.2.9.1)

Since Equation (c.2.9.1) fails to reflect properly the characteristics of K-UHPC in which most of the drying shrinkage occurs rapidly at early age, it is recommended to apply Equation (2.9.2) for K-UHPC. The values of the constant and exponent in Equation (2.9.2) are adjusted by regression analysis based upon drying shrinkage test results of K-UHPC. Besides, unlike ordinary concrete, the significantly slower migration of moisture of K-UHPC owing to its dense structure is reflected in $\epsilon_{ds0}$.

In the equations related to the drying shrinkage, $h$, as the conceptual member

## Specifications

## Commentary

dimension, is expressed by $2A_c/u$ where $A_c$ is the cross-sectional area and $u$ is the circumferential length of the cross-section. In addition, $h_0$ is 100 mm. $t_1$ is a term introduced to express time function in dimensionless form and represents 1-day age. Figure c.2.9.2 plots the drying shrinkage test results of K-UHPC. The drying shrinkage strain ranges between $70\times10^{-6}$~$80\times10^{-6}$ in case of high temperature wet curing, and between $90\times10^{-6}$ ~$100\times10^{-6}$ in case of standard curing.

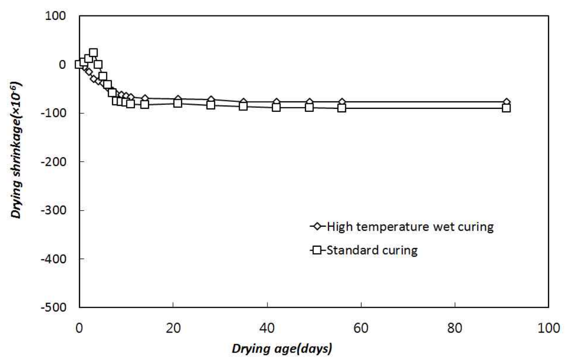



Fig. c.2.9.2 Autogenous shrinkage of K−UHPC

Following, when the conceptual drying shrinkage coefficient is not obtained experimentally, the values of Table c.2.9.1 can be used as conceptual drying shrinkage coefficient according to the age of K-UHPC. These values indicate the conceptual drying shrinkage coefficient for the drying shrinkage that will occur with reference to the age at which the K-UHPC member will form the structural system. For example, this means that, when the structural system is built within 3 days after placing, a value of 100 or 80 should be used for the conceptual drying shrinkage coefficient whereas drying shrinkage can be ignored after 3 months.

## Specifications

(3) The autogenous shrinkage of K-UHPC can be defined by Equations (2.9.3) to (2.9.5).

$$\epsilon_{as}(t) = \epsilon_{as0}(f_{cm})\beta_{as}(t) \tag{2.9.3}$$

$$\epsilon_{as0}(f_{cm}) = \alpha_{as}\left(\frac{f_{cm}/f_{cm0}}{6+f_{cm}/f_{cm0}}\right)^{2.5} \cdot 10^{-6} \tag{2.9.4}$$

$$\beta_{as}(t) = 1-\exp\left[-0.7\left(\frac{t}{t_1}\right)^{0.5}\right] \tag{2.9.5}$$

where $f_{cm}$ is the average compressive strength; $f_{cm0}$= 10 MPa; and, $\alpha_{as}$ is a coefficient of which value is determined by the characteristics of K-UHPC.

## Commentary

Table c.2.9.1 Conceptual drying shrinkage coefficient of K−UHPC ($\times 10^{-6}$)

| Curing condition | Age of K−UHPC | | | |
|---|---|---|---|---|
| | within 3 days | 4~7 days | 28 days | 3 months |
| Standard curing | 100 | 50 | 10 | 0 |
| High temperature wet curing | 80 | 40 | 10 | 0 |

(3) In general, the autogenous shrinkage is known to be more affected by the effects of the mix composition including the adopted materials, W/B ratio and compressive strength rather than the environmental conditions like temperature and humidity. The formulae related to the autogenous shrinkage of K-UHPC adopt the fib Model Code (2010) as basic model. In order to express adequately the autogenous shrinkage characteristics of K-UHPC, modeling is done by modifying the term $\epsilon_{as0}$ related to the final amount of autogenous shrinkage and the term $\beta_{as}$ expressing the characteristics of the development of autogenous shrinkage. $\alpha_{as}$ is determined based upon the autogenous shrinkage test results.

Figure c.2.9.3 plots the autogenous shrinkage test results of K-UHPC. In case of high temperature wet curing, the autogenous shrinkage strain ranges between $450\times10^{-6}$~$500\times 10^{-6}$ and is practically similar in case of standard curing. The autogenous shrinkage occurs rapidly during the curing process in case of high temperature wet curing, but there is practically no difference in the final amount of autogenous shrinkage even when the curing conditions are changed.

Accordingly, when $\alpha_{as}$ is not obtained experimentally, the value of 470 can be used.

**Specifications** | **Commentary**

(4) The total shrinkage of K-UHPC is expressed as Equation (2.9.6) obtained by the summation of Equations (2.9.1) and (2.9.3).

$$\epsilon_{sh}(t,t_s) = \epsilon_{as}(t) + \epsilon_{ds}(t,t_s) \qquad (2.9.6)$$

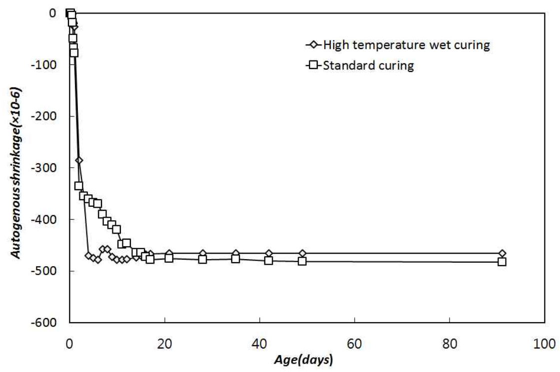


Fig. c.2.9.3 Autogenous shrinkage of K−UHPC

## 2.10 Creep

(1) The creep strain of K-UHPC is proportional to the elastic strain caused by the developed stress and is obtained generally by Equations (2.10.1) and (2.10.2).

$$\epsilon_{cc} = \phi \frac{\sigma_{cp}}{E_{ct}} \qquad (2.10.1)$$

$$\phi(t,t_0) = \phi_0 \beta_c(t,t_0) \qquad (2.10.2)$$

where

$\epsilon_{cc}$ : creep strain of K-UHPC

$\phi$ : creep coefficient

$\sigma_{cp}$ : developed stress

$E_{ct}$ : elastic modulus under loading at age $t$

$\phi_0$ : conceptual creep coefficient

$\beta_c$ : function of age

(2) The creep coefficient of K-UHPC shall be determined considering the effects of the material properties, mix composition, curing

## 2.10 Creep

(1) Similarly to the creep of ordinary concrete, the region in which the creep strain of K-UHPC is proportional to the elastic strain caused by the developed stress occurs generally in the range where the stress is below approximately 40% of the compressive strength.
Even if Equations (2.10.1) and (2.10.2) calculating the creep strain of K-UHPC look similar to those of ACI, these equations express in fact the physical meaning of the creep strain. The results of creep tests performed per age and per curing condition are similar to those given by the formulae of Eurocode and ACI but are closer to ACI known to provide relatively smaller creep strains. These equations express that the creep strain is closer to that of ACI when high temperature wet curing is performed at early age of fabrication in the case of K-UHPC mainly applied to precast members.

(2) The creep of K-UHPC depends on various factors including the material properties and curing conditions. Following, the creep

## Specifications

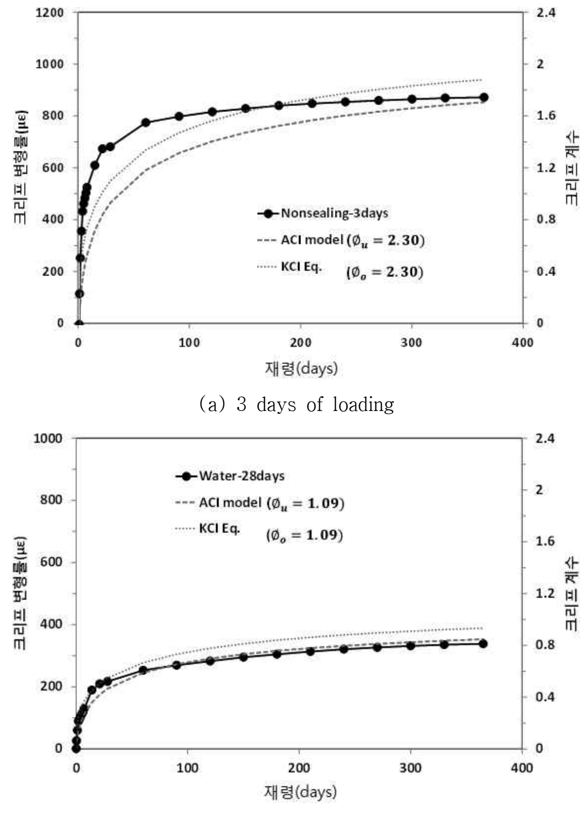
condition, temperature and humidity around the structure and, the shape and dimensions of the member cross-section. In addition, the creep coefficient obtained by compression test is set as standard in the case of tensile stresses lower than the crack strength.

## Commentary

coefficient of K-UHPC used in design must be determined with reference to test results, previous experimental data or to data measured in actual structures.

Besides, since small stress is developed with respect to the creep coefficient in the tension zone, the creep coefficient in the compression zone shall be applied.

In case of standard curing, the creep coefficient varies with the age under loading similarly to ordinary concrete. Figure c.2.10.1 plots the results of creep tests at 3 days and 28 days of loading of K-UHPC that has experienced standard curing.




(a) 3 days of loading

(b) 28 days of loading

**Fig. c.2.10.1** Creep test results for standard curing condition

Accordingly, the final creep coefficient can be set to 1.09 for K-UHPC at 28 days. However, it is recommended to use a final creep coefficient of 2.3 under the specific condition where loading is applied at 3 days.

**Specifications**

(3) In the absence of test, a value of 0.45 shall be used for the conceptual creep coefficient of K-UHPC fabricated using the basic mix composition and wet cured at high temperature.

**Commentary**

(3) Figure c.2.10.2 presents the results of creep tests performed during 1 years on K-UHPC fabricated using the basic mix composition and wet cured at high temperature. As shown in the graph, it appears that the final creep coefficient can be set to 0.45 considering that the creep coefficient at 1 year of loading exceeds merely 0.4 and shows very weak increase after 1 year. This value is more conservative than those proposed by the AFGC-SETRA Recommendations and JSCE Recommendations.

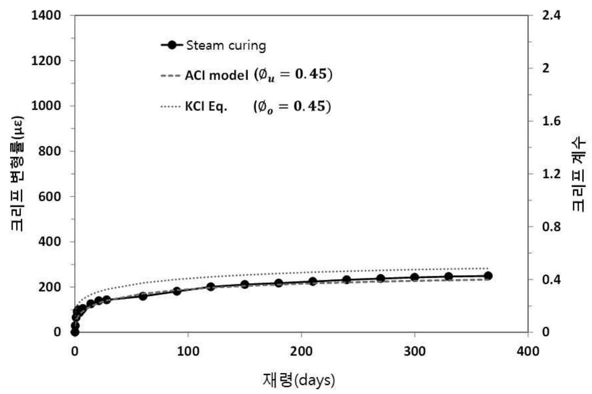



Fig. c.2.10.2 Creep test results for high temperature wet curing condition

**Specifications**

2.11 Fatigue Strength

(1) The characteristic fatigue strength of K-UHPC is obtained based upon the fatigue strength computed from test performed considering the exposure condition of the structure.

(2) In general, the design fatigue strength, $f_{rd}$, for compression and flexural compression of K-UHPC is obtained as function of the stress $\sigma_p$ by means of the fatigue life $N$ and permanent loads using Equation (2.11.1).

$$f_{rd} = 0.85 f_{cd}\left(1 - \frac{\sigma_p}{f_{cd}}\right)\left(1 - \frac{\log N}{17}\right) \qquad (2.11.1)$$

(Note: $N \le 2\times10^6$)

where $f_{cd}$ is the design compressive strength of K-UHPC.

**Commentary**

2.11 Fatigue Strength

(2) Prior to the evaluation of the fatigue strength of K-UHPC by new tests, the fatigue strength of K-UHPC can be evaluated according to the relevant specifications of the JSCE Recommendations.

**Specifications**

(3) The design fatigue strength, $f_{rd}$, for tension and flexural tension of K-UHPC using the basic mix composition and wet cured at high temperature is generally obtained by Equations (2.11.2) to (2.11.4) as function of the stress $\sigma_p$ by means of the fatigue life $N$ and the permanent loads.

For $N \le 7.3\times10^4$:

$$f_{rd} = f_{td}\left(1-\frac{\sigma_p}{f_{td}}\right)\left(1-\frac{\log N}{16}\right) \qquad (2.11.2)$$

For $7.3\times10^4 < N < 1.3\times10^6$:

$$f_{rd} = f_{td}\left(1-\frac{\sigma_p}{f_{td}}\right)\left(2-\frac{\log N}{3.7}\right) \qquad (2.11.3)$$

For $N \ge 1.3\times10^6$:

$$f_{rd} = 0.36 f_{td}\left(1-\frac{\sigma_p}{f_{td}}\right) \qquad (2.11.4)$$

where $f_{td}$ is the design tensile strength of K-UHPC.

(4) The stress, $\sigma_p$, of K-UHPC caused by permanent loads is generally 0 in case of alternating load.

**Commentary**

Chapter 3

# Design Loads

## Specifications

### 3.1 General

(1) In these guidelines, the items related to the design loads and load combinations, strength and general structural analysis used in the design of K-UHPC structural members apply in principle the "Structural Concrete Design Code", except separate regulations like highway bridges, railway bridges and cable-stayed bridges.

(2) The unit weight of the basic mix composition of K-UHPC applied for the computation of the permanent loads is 25.5 kN/m$^3$.

### 3.2 Loads and External Forces

(1) The loads and external forces applied in the design of K-UHPC structures shall comply with the existing relevant design specifications established with respect to the type of structure.

(2) When designing K-UHPC highway bridge, the loads and external forces of the "Highway Bridge Design Code" shall be applied.

(3) When designing K-UHPC railway bridge, the loads and external forces of the "Railway Bridge Design Code" shall be applied.

(4) When designing K-UHPC cable-stayed bridge, the loads and external forces of the

## Commentary

### 3.1 General

### 3.2 Loads and External Forces

(1) Since no need is to define separately the loads and external forces for K-UHPC structures, design can be conducted with respect to the design specifications already established per type of structure.

**Specifications**

**Commentary**

"Steel Cable Stayed Bridge Design Provisions" and "Concrete Cable Stayed Bridge Design Provisions" shall be applied.

## 3.3 Strength

(1) The structure and structural members shall be designed so as to develop design strengths higher than the required strengths calculated using the loads and external loads specified in these guidelines for every cross section.

(2) When computing the compressive strength, flexural strength and shear strength, the design strength shall be computed by applying the material reduction factor and member reduction factor proposed in these guidelines instead of the strength reduction factor.

(3) For other strengths, the "Strength" chapter of the "Structural Concrete Design Code" shall be applied.

## 3.3 Strength (Commentary)

(2) According to the "Strength" chapter of the "Structural Concrete Design Code", the design strength shall be obtained by multiplying the nominal strength by the strength reduction factor. However, due to the fact that it is still difficult to determine clearly the strength reduction factor for K-UHPC, these guidelines calculate conservatively the design strength by applying simultaneously the material reduction factor and the member reduction factor, which are respectively the inverse of the material factor and inverse of the member factor of the JSCE Recommendations, instead of the usual strength reduction factor when calculating the compressive strength, flexural strength and shear strength.

## 3.4 Structural Analysis - General

Structural analysis shall be performed in compliance with the "Structural Analysis - General" chapter of the "Structural Concrete Design Code".

## 3.4 Structural Analysis - General (Commentary)

Chapter 4

# Serviceability and Durability

**Specifications**

## 4.1 General

(1) The serviceability and durability of the K-UHPC structure or structural members shall be examined to ensure that they will maintain sufficiently their function and performances all along their service life when subjected to service loads.

(2) The serviceability check shall be performed considering the effects of cracking, deflection and fatigue.

### 4.1.1 Nomenclature

$f_{ck}$ = characteristic compressive strength of K-UHPC (MPa)

$f_{crd}$ = design crack strength of K-UHPC (MPa)

$f_{crk}$ = characteristic crack strength of K-UHPC (MPa)

$f_{yk}$ = characteristic yield strength of steel (MPa)

$\sigma_l$ = diagonal tension stress (MPa)

$\sigma_x$ = axial stress (MPa)

$\sigma_y$ = stress perpendicular to $\sigma_x$ (MPa)

$\tau$ = shear stress caused by shear force and torsional moment (MPa)

$w_a$ = allowable crack width (mm)

$c_c$ = concrete cover thickness (mm)

**Commentary**

## 4.1 General

### 4.1.1 Nomenclature

**Specifications**

## 4.2 Computation of Stress

The stress of K-UHPC and steel developed in the member cross section during service shall be computed based on the following assumptions.

① The whole cross section of K-UHPC is effective in the computation of stress.

② The strain is proportional to the distance to the neutral axis of the cross section.

③ The elastic moduli of K-UHPC and steel shall comply to those given in Chapter 2.

## 4.3 Limit-Value of Stress

The compressive stress of K-UHPC and tensile stress of steel caused by bending moment and axial force shall not exceed the limit-values expressed in (1) and (2), respectively.

(1) The limit-value of the compressive stress caused by bending moment and axial force is $0.6f_{ck}$ under the application of permanent loads, where $f_{ck}$ is the characteristic compressive strength of K-UHPC.

(2) The limit-value of the tensile stress of steel is $f_{yk}$, where $f_{yk}$ is the characteristic yield strength of steel.

## 4.4 Tensile Stress

(1) This section applies for the examination of the tensile stress caused by bending moment, shear force, torsional moment and axial force.

(2) When examining the serviceability, the tensile stress developed in K-UHPC shall not exceed the design tensile strength.

(3) The computation of the tensile stress

**Commentary**

## 4.2 Computation of Stress

It is necessary to compute the stresses developed in the cross section for the examination during service. K-UHPC shall be assumed as elastic body in the stress computation and tensile stress shall also be considered.

## 4.3 Limit-Value of Stress

(1) When the limit-value is adjusted for K-UHPC subjected to multi-axial constraints, need is to perform evaluation using the appropriate method.

(2) For the examination relative to the fatigue of steel, no need is to limit the tensile stress. However, since this assumption does not hold in the structural analysis and calculation of stress if the tensile stress of steel exceeds the elastic limit, the tensile stress shall be limited below the yield stress.

## 4.4 Tensile Stress

**Specifications**

**Commentary**

caused by bending moment and axial force shall comply with section 4.2. In such case, the limit-value of the tensile stress becomes $f_{td}$, where $f_{td}$ is the design tensile strength.

(4) Equation (4.4.1) shall be applied during the computation of the diagonal tension stress caused by shear force, torsional moment and axial force. In general, the diagonal tension stress shall be examined at the position at centroid of the member cross section where the axial force is 0.

$$\sigma_l = \frac{(\sigma_x + \sigma_y)}{2} + \frac{1}{2}\sqrt{(\sigma_x - \sigma_y)^2 + 4\tau^2} \qquad (4.4.1)$$

where

$\sigma_l$ : diagonal tension stress

$\sigma_x$ : axial stress

$\sigma_y$ : stress perpendicular to $\sigma_x$

$\tau$ : shear stress caused by shear force and torsional moment

(5) The limit-value of the diagonal tension stress caused by shear force, torsional moment and axial force is $f_{td}$, where $f_{td}$ is the design tensile strength.

(6) The shear induced by the shrinkage of K-UHPC shall be considered if necessary during the examination of the tensile stress.

(6) Since K-UHPC experiences larger autogenous shrinkage than ordinary concrete during the hardening, tensile stress is often generated by the restrained strain induced by shrinkage. Accordingly, sufficient measures shall be conceived regard to the constraints due to the forms. For prestressed structures constrained by the tendons disposed inside K-UHPC, it is required to consider the effect of the restrained strain when computing the prestress force. In the absence of prestress in the structure, it is required to verify the safety during service by examining closely the shrinkage provoked by the hardening including the effect of the subsequent presence of residual restrained strain in K-UHPC.

**Specifications**

## 4.5 Displacement and Deformation

The "Deflection" chapter of the "Structural Concrete Design Code" shall be applied.

## 4.6 Vibrations

The absence of loss of the function and serviceability of the structure due to the vibrations induced by varying loads shall be demonstrated by means of appropriate methods.

## 4.7 Fatigue

### 4.7.1 General

Fatigue check shall be performed using the fatigue strength given in section 2.11.

**Commentary**

## 4.5 Displacement and Deformation

Since these guidelines specify that the crack width in service width shall not exceed in principle the allowable crack width, the displacement and deformation under service load shall be computed considering the stress-crack width relationship of K-UHPC. Other items shall comply in principle with the "Structural Concrete Design Code". Attention shall be paid on the fact that larger displacement and deformation may occur in the K-UHPC structure compared to ordinary concrete structures due to the thinner member thickness.

## 4.6 Vibrations

Attention shall be paid on the fact that the structure using K-UHPC exhibits longer fundamental period than ordinary concrete structures due to its thinner member thickness.

## 4.7 Fatigue

### 4.7.1 General

Structures using K-UHPC generally apply thinner member thickness. This implies that the self weight is reduced and that the structure experiences relatively larger varying loads than permanent loads. Therefore, it is necessary to investigate fatigue considering such feature of the structures using K-UHPC. Even if fatigue check shall generally be performed on steel for cyclic loads generated by bending moments, fatigue check shall also be conducted on K-UHPC if necessary.
Besides, according to the JSCE Recommendations, the pore water pressure in the concrete of ordinary concrete structures increases due to cyclic stresses when concrete is in underwater or in moisture condition. This

**Specifications**

**Commentary**

leads to the acceleration of the propagation of cracks and results to the loss of the fatigue strength to approximately 2/3~ 4/5 of the fatigue strength in dry air state. However, the extremely dense structure of the matrix in K-UHPC annihilates nearly completely the effect of the pore water pressure. Accordingly, the JSCE Recommendations state and verify that the loss of fatigue strength shall not be considered even in underwater and moisture condition.

## 4.8 Serviceability Check

(1) Except for those cases where the allowable crack width is specified and cracking is examined like structures requiring special watertightness or structures in which aesthetics is important, crack check is assumed to be completed when all the regulations of these guidelines are satisfied.

(2) The allowable crack widths of Table 4.8.1 shall be respected when serviceability check is performed for structures experiencing cracking during construction or after completion. When the crack width is examined analytically, Equation (4.8.1) shall be satisfied. The calculation of the crack width by analysis shall follow the method presented in Appendix 6 of these guidelines.

$$w \le w_a \tag{4.8.1}$$

## 4.8 Serviceability Check

(1) As explained in the Commentary section 2.4.4, these guidelines apply the model in which K-UHPC reaches the tensile strength at crack width of 0.3 mm in the design. Following, in case where design is performed in compliance with these guidelines, it can basically be assumed that the crack width remain below 0.3 mm and, therefore, that separate crack check is not required.

(2) The French AFGC-SETRA Recommendations for the design of ultra high performance concrete propose allowable crack widths for serviceability check based upon the French design codes BAEL and BPEL dedicated to steel reinforced concrete structures abd prestressed concrete structures. The proposed allowable crack widths are 0.3 mm in normal environment, 0.2 mm in unfavorable environment, and 0.1 mm in very unfavorable environment. Accordingly, the allowable crack widths for the serviceability check of K-UHPC structures refer to those of the "Structural Concrete Design Code" for ordinary concrete structures.

## Specifications

## Commentary

**Table 4.8.1** Allowable crack width $w_a$ of K-UHPC structure (mm)

| Type of steel | Environmental condition related to steel corrosion | | | |
|---|---|---|---|---|
| | Dry air | Moisture | Corrosive | Highly corrosive |
| Rebar | largest value between 0.4 mm and $0.006c_c$ | largest value between 0.3 mm and $0.005c_c$ | largest value between 0.3 mm and $0.004c_c$ | largest value between 0.3 mm and $0.0035c_c$ |
| PS steel | largest value between 0.2 mm and $0.005c_c$ | largest value between 0.2 mm and $0.004c_c$ | – | – |

Chapter 5

# Structure Details

## Specifications

### 5.1 General

This chapter specifies the structure details for the design of structures using K-UHPC. Moreover, when structure details are specified per structure and type of member, these structure details shall also be satisfied.

### 5.2 Minimum Cover Thickness

The minimum rebar cover thickness of structures made of K-UHPC and used in normal environment shall be the largest value between 20 mm and 1.5 times the diameter of the reinforcing steel. In case of thinner cover thickness, test shall be performed to verify the satisfactory filling during placing of K-UHPC, and the durability and safety of the structure.

## Commentary

### 5.1 General

Structure details are also specified in other chapters. These structure details shall be applied only on the members and structures in the relevant chapter. The structure details specified in each other chapter prevail to those specified in this chapter. In addition, the structure details of this chapter apply under the condition of standard construction.

### 5.2 Minimum Cover Thickness

The minimum cover thickness is determined mainly with regard to the workability and considering also the durability and safety. Moreover, construction experiences in Korea and foreign countries are also referred.

K-UHPC exhibits improved durability owing to its outstanding characteristics related to the internal migration of substances thanks to the densification of its structure compared to usual concrete. Compared to the diffusion coefficient of chloride ion of $7.3\times10^{-12}$ ㎡/sec in ordinary concrete with compressive strength of about 30 MPa and, of $0.8\times10^{-12}$ ㎡/sec in high strength concrete with compressive strength of about 50 MPa, the diffusion coefficient of chloride ion in K-UHPC is significantly smaller with a value running

**Specifications**

**Commentary**

around $0.02 \times 10^{-12}$ ㎡/sec. Such characteristics is an important factor enabling to design thinner cover thickness for K- UHPC than for ordinary concrete.

Since K-UHPC develops better anchor performance than ordinary concrete, pull out tests of rebar were conducted to determine the cover thickness enabling to drive the utmost performance of the steel reinforcement. Figure c.5.2.1 arranges the corresponding results and plots the variation of the maximum bond load per type of rebar during the pull out of rebar according to the cover thickness. It appears that the maximum bond load, that is the bond performance enabling to reach the yield load of the rebar, is already developed when the cover thickness is less than twice the diameter of the rebar. If the serviceability limit is set prior to the yield of the rebar, it is reasonable to limit the cover thickness to 1.5 times the diameter of the rebar rather than twice the diameter of the rebar.

In case where a cover thickness apart from this regulation is applied, the satisfactory filling during placing of K-UHPC simultaneously with the satisfaction of the required durability shall be verified by test and analysis together with the execution of bond test to verify if the safety as structure is secured. Since bond failure may occur in prestressed members when using thin cover thickness, attention shall be paid on that matter.

In addition, the minimum cover thickness is significantly smaller in structures using K-UHPC than in structures using ordinary concrete. Following, attention shall be paid to keep construction errors as small as possible since they may have relatively larger effect.

**Specifications** | **Commentary**

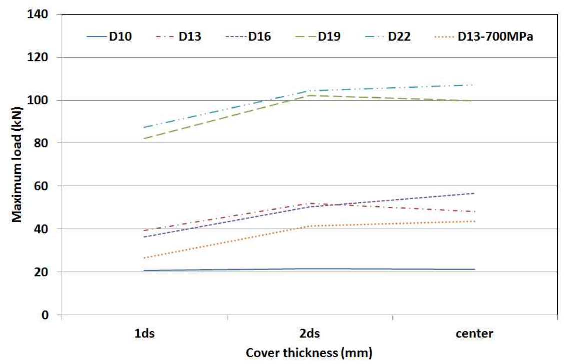



Fig. c.5.2.1 Variation of peak load sustained by rebar according to cover thickness

## 5.3 Spacing of Reinforcing Steel

The minimum spacing of the reinforcing steel in structures using K-UHPC shall be larger than 20 mm or the diameter of the reinforcing steel. In case of shorter spacing, test shall be performed to verify the satisfactory filling during placing of K-UHPC, and the durability and safety of the structure.

## 5.3 Spacing of Reinforcing Steel

The minimum spacing of the reinforcing steel like rebar and tendon considers the workability and refers to construction achievements and experimental results obtained in Korea and foreign countries. Since cracking or failure were not observed until yielding of the reinforcing steel in the bond tests of K-UHPC performed using reinforcing steel spacing 1 and 2 times the diameter of the reinforcing steel, the minimum spacing of the reinforcing steel is set to the diameter of the reinforcing steel. In case of smaller spacing, it is required to verify if satisfactory filling is secured during the placing of K-UHPC.

Chapter 6

# Flexure and Axial Loads

**Specifications**

## 6.1 General

The regulations of this chapter apply for the design of members subjected to bending moment or axial force or, members subjected simultaneously to bending moment and axial force.

### 6.1.1 Nomenclature

$A_c$ = cross-sectional area of K-UHPC ($mm^2$)

$f_{cd}$ = design compressive strength of K-UHPC (MPa)

$k_1$ = strength reduction factor considering the difference in strength between specimen and structure, 0.85

$M_d$ = design moment (N . mm)

$N_d$ = design compression (N)

$N_{oud}$ = design compressive strength of member subjected to axial compression (N)

$\phi_b$ = member reduction factor

## 6.2 Compressive Strength

The design compressive strength, $N_{oud}$, of the member subjected to axial compression shall be computed using Equation (6.2.1).

$$N_{oud} = \phi_b k_1 f_{cd} A_c \tag{6.2.1}$$

**Commentary**

## 6.1 General

### 6.1.1 Nomenclature

## 6.2 Compressive Strength

Despite of the existence of a method considering the effect of steel arranged in the axial direction and direction perpendicular to the axis of the member, this method is ignored in the equation computing the

**Specifications**

where

$A_c$ : cross-sectional area of K-UHPC

$f_{cd}$ : design compressive strength of K-UHPC

$k_1$ : 0.85

$\phi_b$ : member reduction factor, generally 0.77

**Commentary**

compressive strength, $N_{oud}$, for computational convenience.

$k_1$ is a strength reduction factor to reflect the case where the compressive strength of the structural body subjected to bending and axial force is smaller than that of the specimen. For ordinary concrete, $k_1$ shall be reduced according to the increase of the strength. One reason for that is that splitting of the concrete cover caused by the effect of shrinkage occurs before compressive failure in the steel reinforced member using high strength concrete and subjected to compression. However, there is poor probability of splitting in K-UHPC owing to the steel fiber reinforcement. Therefore, the value $k_1 = 0.85$ used for ordinary concrete can be adopted.

When $M_d/N_d$ is small in the axially loaded member, the member reduction factor is set to 0.77 by considering the necessity to provide an upper limit of the strength in compression in order to prepare for unexpected eccentricity during construction.

## 6.3 Flexural Strength

(1) The following assumptions ①~③ shall be adopted when computing the design flexural strength of the member subjected to bending moment and simultaneously to bending moment and axial force for the member section force or the member cross sectional width according to the direction of the section force. In such case, the member reduction factor, $\phi_b$, is generally 0.91.

① The strain is proportional to the distance from the neutral axis of the cross section.

② The stress-strain curve of K-UHPC follows chapter 2.

③ The stress-strain curve of steel complies

## 6.3 Flexural Strength

(1) Ultra high performance steel reinforced concrete can sustain tensile stresses in stability even after cracking owing to the bridging action of the steel fibers. Since the contribution of the tensile stress to the flexural strength is significantly higher than in ordinary concrete, it is reasonable to consider the tensile stress in the computation of the flexural strength.

Figure c.6.3.1 and Table c.6.3.1 compare the flexural strength obtained by test on 12 rectangular K-UHPC beams with dimensions of 0.18 m×0.27 m

and 4 T-shape K-UHPC beams with heights varying from 0.6 m to 1.3 m to the

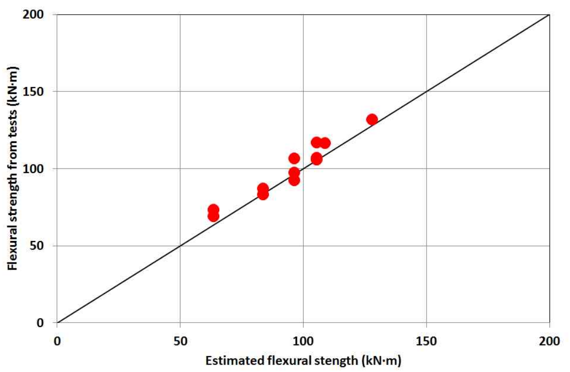
## Specifications

with the “Structural Concrete Design Code”.

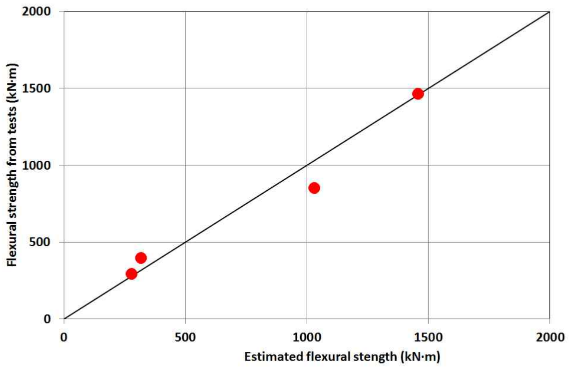
## Commentary

corresponding flexural strength predicted using the flexural moment computation method of section 6.3(1). Here, the compressive and tensile behaviors used for the predictions are those obtained through material test and not the design material properties. The average ratio of the predicted value to the test value is 0.97, which indicates that the flexural strength of the K-UHPC beam provided by the flexural moment computation method of section 6.3(1) achieves reasonable prediction.

Considering that the material properties proposed in chapter 2 underestimate the actual material properties accounting for the safety ratio, the flexural strength evaluated by applying the method of this section can be accepted as conservative.




(a) Test results for rectangular beam




(b) Test results for T-shape beam

Fig. c.6.3.1 Comparison of predicted and experimental flexural strengths

**Specifications** | **Commentary**

Table c.6.3.1 Comparison of predicted and experimental flexural strengths

| Member | | Flexural strength | | Prediction/ Test |
|---|---|---|---|---|
| | | Prediction (kN·m) | Test (kN·m) | |
| T-shape beam | T600NS | 280.1 | 293.1 | 0.96 |
| | T600 S | 319.2 | 395.0 | 0.81 |
| | T1000S | 1031.6 | 850.8 | 1.21 |
| | T1300S | 1458.6 | 1462.3 | 1.00 |
| Shape of member: T-shaped beam<br>Effective depth: 600 mm, 1,000 mm, 1,300 mm<br>Reinforcing steel: none(NS), present(S) | | | | |
| Rectangular beam | NRE-1 | 63.5 | 68.8 | 0.92 |
| | NRE-2 | 63.5 | 73.1 | 0.87 |
| | R12E-1 | 83.6 | 87.0 | 0.96 |
| | R12E-2 | 83.6 | 83.3 | 1.00 |
| | R13E-1 | 96.3 | 97.5 | 0.99 |
| | R13E-2 | 96.3 | 106.5 | 0.90 |
| | R13M-1 | 96.3 | 92.2 | 1.04 |
| | R14E-1 | 108.8 | 116.5 | 0.93 |
| | R14E-2 | 105.4 | 116.8 | 0.93 |
| | R22E-1 | 105.4 | 107.0 | 0.99 |
| | R22E-2 | 105.4 | 105.7 | 1.00 |
| | R23E-1 | 128.0 | 131.7 | 0.97 |
| Average | | | | 0.97 |
| Cross section shape: rectangular beam (180 mm×270 mm)<br>Reinforcing steel: none(NR), present(R)<br>Rebar arrangement: 1-row(R1), 2-row(R2)<br>Placing position: mid-span(M), end(E) | | | | |

(2) The effect of second-moment shall be considered if necessary.

(2) Since the member using K-UHPC tends generally to show larger slenderness ratio than that using ordinary concrete, the effect of the second-moment shall be considered if necessary. Detailed calculation shall comply with the “Structural Concrete Design Code”.

## 6.4 Structure Details

No particular regulation is provided for the minimum amount of steel.

## 6.4 Structure Details

K-UHPC can withstand tensile stress owing to the bridging effect of the steel fibers even after cracking and does not experience brittle fracture. Therefore,
no particular regulation is provided for the minimum amount of steel.

Chapter 7

# Shear and Torsion

**Specifications**

## 7.1 General

(1) The regulations of this chapter shall be applied for the shear and torsion design of K-UHPC members.

(2) The minimum rebar arrangement for shear or torsional load can be omitted for K-UHPC members.

### 7.1.1 Nomenclature

$A_0 =$ area delimited by the centerline of shear flow (mm$^2$)

$A_0^{real} =$ area delimited by the centerline of shear flow acting on $t_{real}$ (mm$^2$)

$A_{cp} =$ area of the K-UHPC section in contact with the exterior (mm$^2$)

$A_g =$ total section area (mm$^2$)

$A_{sl} =$ cross sectional area of longitudinal rebar of the ultra high performance concrete beam necessary to support

**Commentary**

## 7.1 General

(2) Brittle fracture does not occur in the K-UHPC member even in the absence of minimum rebar arrangement because the steel fibers in K-UHPC play the role fulfilled by the minimum rebar arrangement in ordinary concrete structure. In other words, unpredicted brittle failure does not occur due to the prevention or delay of the propagation of cracks owing to the bridging action of the steel fibers even under the occurrence of diagonal tensile cracks induced by shear or torsion.

### 7.1.1 Nomenclature

**Specifications**

**Commentary**

$T_d$ (mm²)

$A_v =$ cross sectional area of 1 strand of closed stirrup resisting to torsion within spacing $s$ (mm²)

$A_w =$ cross sectional area of shear stiffener disposed at spacing $s_s$ (mm²)

$b_w =$ width of web (mm)

$d =$ effective depth (mm)

$f_{cd} =$ design compressive strength of K-UHPC (MPa)

$f_{crd} =$ design crack strength of K-UHPC (MPa)

$f_{ld} =$ design yield strength of longitudinal rebar (MPa)

$f_{pc} =$ compressive stress of K-UHPC considering all the prestress loss at the center of the section resisting to the acting loads or, compressive stress at the intersection between the web and flange when the center of the section is located in the flange (MPa)

$f_{td} =$ design tensile strength of K-UHPC (MPa)

$f_{tk} =$ characteristic tensile strength of K-UHPC (MPa)

$f_v =$ average tensile strength (MPa)

$f_{vd} =$ design average tensile strength in the direction perpendicular to the diagonal tensile crack of K-UHPC (MPa)

$f_{yvd} =$ design yield strength of lateral reinforcement or shear stiffener (MPa)

$f_{yld} =$ design yield strength of longitudinal reinforcement (MPa)

$P_{cp} =$ circumferential length of $A_{cp}$ (mm)

$P_{ed} =$ effective tension of longitudinal tendon (N)

$p_h =$ circumferential length of $A_0$ (mm)

$s =$ spacing of lateral reinforcement acting effectively as torsional reinforcement (mm)

$s_s =$ spacing of shear stiffener arrangement

**Specifications**

(mm)

$t =$ thin-wall thickness when assuming the K-UHPC beam as equivalent thin-walled tube (mm), $t = 2A_{cp}/3P_{cp}$

$t_{real} =$ actual wall thickness of hollow section (mm)

$T_d =$ design torsional strength of K-UHPC (N . mm)

$T_{K-UHPC} =$ torsional strength induced by K-UHPC (N . mm)

$T_s =$ torsional strength induced by closed stirrup (N . mm)

$T_u =$ factored torsional moment (N . mm)

$u_p =$ circumferential length of design cross section, computed at the position located at a distance $d/2$ from the loaded face (mm)

$V_{fd} =$ shear strength sustained by steel fiber (N)

$V_{pcd} =$ punching shear strength in case of local loading acting on the plane member and occurrence of punching shear failure (N)

$V_{ped} =$ component of effective tension of longitudinal tendon parallel to shear force (N)

$V_{rpcd} =$ shear strength of beam member that does not use shear stiffener excluding the contribution of steel fiber (N)

$V_d =$ design shear strength of K-UHPC (N)

$V_{sd} =$ shear strength induced by shear stiffener (N)

$V_{wcd} =$ diagonal compressive strength relative to shear of web concrete (N)

$w_u =$ crack width at maximum stress (mm)

$z =$ distance from the position of the resultant of the compressive stresses to the centroid of tensile steel (mm), generally $d/1.15$

$\alpha_p =$ angle between the longitudinal stiffener and the axis of the member

**Commentary**

**Specifications**

$\alpha_s =$ angle formed by shear stiffener with longitudinal axis of beam

$\beta_o =$ angle at which the diagonal tensile crack is inclined by 45° to the axis of the member in the absence of axial force

$\beta_u =$ angle occurring between axial direction and diagonal tensile crack plane, larger than 30°

$\phi_b =$ member reduction factor

$\theta =$ inclination angle of compression strut (45° in absence of prestress)

$\sigma_{xu} =$ average compressive stress in axial direction (MPa)

$\sigma_{yu} =$ average compressive stress in direction perpendicular to axial direction (MPa)

$\sigma(w) =$ stress of tension-softening curve (MPa)

$\tau =$ average shear stress caused by design section force (MPa)

## 7.2 Shear

### 7.2.1 Shear Strength of Beam Member

(1) The design shear strength, $V_d$, is obtained by Equation (7.2.1).

$$V_d = V_{rpcd} + V_{fd} + V_{sd} + V_{ped} \qquad (7.2.1)$$

where $V_{rpcd}$ is the shear strength of the beam member that does not use shear stiffener, excluding the contribution of steel fiber and, obtained by means of Equation (7.2.2).

$$V_{rpcd} = \phi_b\left(0.18\sqrt{f_{cd}}\,b_w d\right) \qquad (7.2.2)$$

where

$b_w$ : width of web

$d$ : effective depth

$f_{cd}$ : design compressive strength of K-UHPC

$\phi_b$ : member reduction factor, generally 0.77

**Commentary**

## 7.2 Shear

### 7.2.1 Shear Strength of Beam Member

(1) The shear strength formula of beam member was proposed based on the same concept as the JSCE Recommendations.

The shear strength $V_d$ is expressed as the sum of the contribution $V_{rpcd}$ of the matrix of K-UHPC, the contribution $V_{fd}$ of the steel fibers, the contribution $V_{sd}$ of the stirrup (in presence of stirrup) and the component $V_{ped}$ parallel to the shear force of the axial tendon. $V_{rpcd}$ is the shear strength developed by the interaction of the stress transfer caused by the interlocking in the crack surface, the stress transfer in the compression zone and, the dowel action in the tension steel between the matrix and the steel fibers. $V_{fd}$ is the shear strength caused by the average tensile strength developed in the diagonal tensile cracks of the K-UHPC member.

## Specifications

The shear strength sustained by the steel fiber, $V_{fd}$, is obtained by Equation (7.2.3).

$$V_{fd} = \phi_b (f_{vd} / \tan\beta_u) b_w z \qquad (7.2.3)$$

where

$f_{vd}$ : design average tensile strength in the direction perpendicular to diagonal tensile crack of K-UHPC

$\beta_u$ : angle occurring between axial direction and diagonal tensile crack plane. This angle shall be larger than 30°.

$$\beta_u = \frac{1}{2} tan^{-1}\left(\frac{2\tau}{\sigma_{xu} - \sigma_{yu}}\right) - \beta_o$$

where

$\tau$ : average shear stress caused by design section force

$\sigma_{xu}$, $\sigma_{yu}$ : average compressive stresses in axial direction and direction perpendicular to axis, respectively

$\beta_o$ : angle at which the diagonal tensile crack is inclined by 45° to the axis of the member in the absence of axial force

$z$ : distance from the position of the resultant of the compressive stresses to the centroid of tensile steel (mm), generally $d/1.15$.

$\phi_b$ : member reduction factor, generally 0.77

$V_{sd}$ is the shear strength caused by the shear stiffener and is generally obtained by Equation (7.2.4).

$$V_{sd} = \phi_b \frac{A_w f_{yvd} (\sin\alpha_s + \cos\alpha_s)}{s_s} d \qquad (7.2.4)$$

where

$A_w$ : cross sectional area of shear stiffener disposed at spacing $s_s$

$f_{yvd}$ : design yield strength of shear stiffener

$\alpha_s$ : angle formed by shear stiffener with longitudinal axis of beam

## Commentary

The design average tensile strength, $f_{vd}$, in the direction perpendicular to the diagonal tensile crack of K-UHPC can be assessed to be closely related to the tension-softening characteristics of K-UHPC. In the AFGC-SETRA Recommendations, the average tensile strength $f_v$ is defined as the average of the tensile stress up to $w_v$ in the tension-softening curve $\sigma(w)$, as expressed in Equation (c.7.2.1).

$$f_v = \frac{1}{K_f}\frac{1}{w_v}\int_0^{w_v} \sigma(w)dw \qquad (c.7.2.1)$$

where

$w_v = \max(w_u,\ 0.3\mathrm{mm})$

$\sigma(w)$ : tension-softening curve

$K_f$ : in general 1.25

$w_u$ : crack width at peak stress

In these guidelines, the design average tensile strength in the direction perpendicular to diagonal tensile crack K-UHPC can be expressed as Equation (c.7.2.2) since the material reduction factor considers the orientation of the steel fibers.

$$f_{vd} = \frac{1}{w_v}\int_0^{w_v} \phi_c \sigma_k(w)dw = \frac{1}{w_v}\int_0^{w_v} \sigma_d(w)dw \qquad (c.7.2.2)$$

where

$w_v = \max(w_u,\ 0.3\mathrm{mm})$

$\phi_c$ : material reduction factor, 0.8

$\sigma_k(w)$ : tension-softening curve of Figure 2.4.1

$\sigma_d(w)$ : value obtained by multiplying the material reduction factor to $\sigma_k(w)$

$\beta_o$ can be set to $\beta_o = 5°$ based upon previous experimental results.

The shear strength provided by these guidelines is compared to that obtained through shear test of K-UHPC beams (Table c.7.2.1). The shear strength provided by these guidelines and considering redundantly the material reduction factor and the member

## Specifications

$s_s$ : spacing of shear stiffener arrangement
$\phi_b$ : member reduction factor, generally 0.91
$d$ : effective depth

$V_{ped}$, the component of the effective tension of the longitudinal tendon parallel to shear force, is obtained by Equation (7.2.5).

$$V_{ped} = \phi_b P_{ed} \sin\alpha_p \qquad (7.2.5)$$

where

$P_{ed}$ : effective tension of longitudinal tendon
$\alpha_p$ : angle between the longitudinal stiffener and the axis of the member
$\phi_b$ : member reduction factor, generally 0.91

## Commentary

reduction factor accounts approximately for 60% on the mean of the actual shear strength obtained from test. In general, the "Structural Concrete Design Code" or the ACI Design Code (2011) specify an adequate safety factor for shear of approximately 70% considering the brittle failure mode. It appears thus that these guidelines secure sufficient factor of safety.

Table c.7.2.1 Verification of shear strength evaluation formula

| Member※ | Experimental value (kN) | Design value considering the material reduction factor and member reduction factor (kN) | Design /Experiment |
|---|---|---|---|
| | A | B | B/A |
| S25-F20-P0 | 526.5 | 263.4 | 0.5 |
| S34-F20-P0 | 404.0 | 263.4 | 0.65 |
| S25-F20-PS | 716.5 | 358.9 | 0.5 |
| S34-F20-PS | 477.0 | 358.9 | 0.75 |
| Average | | | 0.60 |

※ Effective depth 640 mm, Shear span-to-depth ratio: 2.5(S25), 3.5(S34), F20 : Steel fiber 2%, Prestressing: none(P0, $\beta_u$ = 40°), present(PS, $\beta_u$ =30°)

For information, in order to consider rationally the effect of the orientation of the steel fibers by applying the strength reduction factor ($\phi < 1$, $\phi_c = \phi_b = 1.0$) while securing an appropriate safety factor of approximately 70%, the strength reduction factor ($\phi_f$) becomes 0.66 by assuming a steel fiber orientation factor of 1.14 to which the shear reduction factor ($\phi$) of 0.75 of the "Structural Concrete Design Code" is applied additionally. The application of this factor enables to secure a shear strength corresponding to 64% on the mean of the design strength (Table c.7.2.2).

## Specifications

(2) The diagonal compressive strength relative to the shear of web concrete, $V_{wcd}$, is obtained by Equation (7.2.6).

$$V_{wcd} = \phi_b \left[ 0.84 f_{cd}^{2/3} \sin(2\beta_u) b_w d \right] \qquad (7.2.6)$$

where

$\phi_b$ : member reduction factor, generally 0.77

7.2.2 Punching Shear Strength

(1) In case of local loading acting on the plane member, the design punching shear strength can be obtained by Equation (7.2.7).

$$V_{pcd} = \phi_b f_{vd} u_p d \qquad (7.2.7)$$

where

$f_{vd}$ : design average tensile strength in the direction perpendicular to the diagonal tensile crack of K-UHPC

$d$ : effective depth

$u_p$ : circumferential length of design cross section, computed at the position located at a distance $d/2$ from the

## Commentary

Table c.7.2.2 Verification of formula proposed in K-UHPC design recommendations

| Member※ | Experimental value (kN) | Analytic value applying $\phi_f = 0.66$, $\phi_c = \phi_b = 1.0$ | | Analysis/Experiment | |
|---|---|---|---|---|---|
| | | $\beta_u = 30^\circ$ | $\beta_u = 40^\circ$ | | |
| | A | B | C | B/A | C/A |
| S25-F20-P0 | 526.5 | | 277.5 | | 0.53 |
| S34-F20-P0 | 404 | | 277.5 | | 0.69 |
| Average | | | | | 0.61 |
| S25-F20-PS | 716.5 | 380.0 | | 0.53 | |
| S34-F20-PS | 477 | 380.0 | | 0.80 | |
| Average | | | | 0.66 | |

※ Effective depth 640 mm, Shear span-to-depth ratio: 2.5(S25), 3.5(S34), F20 : Steel fiber 2%, Prestressing: none(P0), present(PS)

(2) As a case of shear failure mode, there is the failure of the member through compression failure of the web concrete. This article intends to prevent such failure pattern. In Equation (7.2.6), $f_{cd}$ is proportional to the 2/3 power of the compressive strength without fixing any upper bound. In addition, the compression acting on the web concrete is computed as expressed in Equation (7.2.6) considering the equilibrium conditions of the forces based upon the angle of inclination of the diagonal tensile crack.

7.2.2 Punching Shear Strength

(1) $f_{vd}$ is obtained by Equation (c.7.2.2). In the case of K-UHPC deck, tensile steel is often unnecessary. The effective depth $d$ in such case can be assumed to be 80% of the thickness of the deck.

**Specifications**

loaded face.

$\phi_b$ : member reduction factor, generally 0.77

## 7.3 Torsion

### 7.3.1 General

(1) In case where securing the static equilibrium for torsion is imperatively required for the safety of the structural element in the ultra high performance concrete structure, check for torsion shall be performed. Besides, check for torsion can be skipped for the element member of the statically indeterminate structure for which torsion is not governing the safety of the structure.

(2) In the case of ultra high performance concrete structures in which the steel fiber plays the role of minimum reinforcement of ordinary concrete, no need is to arrange additionally lateral and longitudinal minimum reinforcement since the crack width remains extremely small (generally smaller than 0.3 mm) even if cracking occurs under service load.

(3) The torsional strength of the cross section shall be computed based upon the adequate thin-wall theory. The plain section is replaced by an equivalent thin-walled tube to compute the torsional strength. For complicated cross sections like the T-shape cross section, the cross section shall be divided into a series of element cross-sections to calculate the torsional strength of each element cross section and, the torsional strength of the original cross section shall be obtained by summing up the strengths of each element.

**Commentary**

## 7.3 Torsion

### 7.3.1 General

(3) The ultra high performance concrete beam subjected to torsion can be idealized by an equivalent thin-walled tube neglecting a part of the internal concrete as shown in Figure c.7.3.1. If cracking occurs in the ultra high performance concrete member due to torsion, the torsional resistance is determined by the closed stirrup, the longitudinal reinforcement arranged near the surface, and by the ultra high performance concrete transferring the load even after cracking. The multiplication of the shear stress ($\tau$) at a point on the centerline of the closed tube wall and the idealized wall thickness ($t$) gives the shear flow $q = \tau t$. The shear flow caused by torsion is constant in all the points of the circumference and runs along the centerline of the wall. The shear stress caused by torsion at

**Specifications**

(4) The torsional moment generated by the load acting in each element cross section of complicated cross sections like the T-shape cross-section can be shared proportionally to the torsional rigidity of the uncracked section of each element. Verification for safety shall be performed independently for each subdivided element cross-section.

7.3.2 Design

(1) The effect of torsion can be neglected for the following cases.

Case without prestress:

$$T_u < f_{crd}\frac{2A_{cp}^2}{9P_{cp}} \tag{7.3.1}$$

Case with prestress:

$$T_u < f_{crd}\frac{2A_{cp}^2}{9P_{cp}}\sqrt{1+\frac{f_{pc}}{f_{crd}}} \tag{7.3.2}$$

Case subjected to longitudinal tension and compression:

$$T_u < f_{crd}\frac{2A_{cp}^2}{9P_{cp}}\sqrt{1+\frac{N_u}{A_g f_{crd}}} \tag{7.3.3}$$

$T_u$ : factored torsional moment

**Commentary**

an arbitrary point on the centerline of the wall can be expressed as $\tau = T/(2A_0 t)$, where $A_0$ is the total area delimited by the shear flow. In case of a hollow section, the inner hollow area shall also be included.

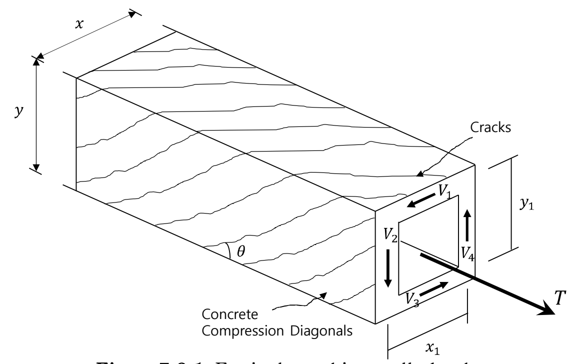


Fig. c.7.3.1 Equivalent thin-walled tube

7.3.2 Design

(1) If the factored torsional moment $T_u$ is smaller than 1/4 of the cracked torsional moment $T_{cr}$, $T_u$ has no significant effect on the flexural and shear strengths and can be neglected. According to the results of tests for cracked torsional moment performed on a total of 17 K-UHPC beams (cross-section: 300 mm×300 mm(14 beams), 300 mm×470 mm(3 beams), stirrup spacing of 0, 100, 200 mm, use of actual values obtained through tensile test instead of design tensile stress) and arranged in Table c.7.3.2, the cracked torsional moment is derived by idealizing the beams by means of thin-walled tubes with thickness $t = 2A_{cp}/3P_{cp}$ and circumferential area $A_0 = 2A_{cp}/3$ prior to cracking and, assuming

## Specifications

$A_{cp}$ : area of the K-UHPC section in contact with the exterior

$P_{cp}$ : circumferential length of $A_{cp}$

$A_g$ : total cross-sectional area

(2) The design torsional strength of the K-UHPC beam is obtained by Equation (7.3.4). Here, the required amount of longitudinal reinforcement is given by Equation (7.3.5).

$$T_d = T_s + T_{K-UHPC} \quad (7.3.4)$$

$$= \phi_b\left(\frac{2A_0 A_v f_{yvd}}{s}\cot\theta\right) + \phi_b\left(2A_0 f_{td} t \cot\theta\right)$$

$$A_l = \frac{A_v}{s}\left(\frac{f_{yvd}}{f_{yld}}\right)p_h \cot^2\theta - \frac{f_{td} t p_h}{f_{yld}}\left(1-\cot^2\theta\right) \quad (7.3.5)$$

where

$T_d$ : design torsional strength of K-UHPC beam

$T_s$ : torsional strength induced by closed stirrup

$T_{K-UHPC}$ : torsional strength induced by K-UHPC

$A_0$ : area delimited by the centerline of shear flow

$A_v$ : cross sectional area of 1 strand of closed stirrup resisting to torsion within spacing $s$

## Commentary

that cracking occurs when the principal stress reaches the design crack stress $f_{crd}$.

$$T_{cr} = f_{crd}\frac{8A_{cp}^2}{9P_{cp}} \quad (c.7.3.1)$$

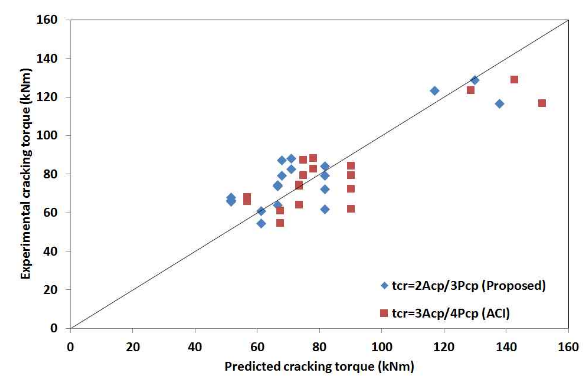



Fig. c.7.3.2 Comparison of cracked torsional moment of K-UHPC beam

The increase of the cracked torsional moment of the prestressed member does not differ from the ordinary concrete structure.

(2) The torsional resistance of the ultra high performance concrete beam can be obtained by integrating the thin-walled tube theory and the properties of ultra high performance concrete enabling to transfer tension even after cracking. The shear force ($V_2$) generated by the factored torsional moment and developed on one face of the beam is supported by the vertical force ($\Sigma A_t f_{yv}$) occurring in the closed stirrup and the vertical component of the tension ($\Sigma f_t t$) transferred in the cracked ultra high performance concrete surface (Fig. c.7.3.3). Here, the longitudinal force developed simultaneously by the factored torsional moment is supported by the longitudinal reinforcement and the horizontal component of the tension transferred in the cracked ultra high performance concrete surface (Fig. c.7.3.4).

## Specifications

$s$ : spacing of closed stirrup

$t$ : thin-wall thickness when assuming the K-UHPC beam as equivalent thin-walled tube, $t = 2A_{cp}/3P_{cp}$

$\theta$ : inclination angle of compression strut (45° in case without prestress)

$A_{sl}$ : cross sectional area of longitudinal rebar of the ultra high performance concrete beam necessary to support $T_d$

$p_h$ : circumferential length of $A_0$ (mm)

$f_{td}$ : design tensile strength of K-UHPC

$f_{yvd}$ : design yield strength of lateral reinforcement

$f_{yld}$ : design yield strength of longitudinal reinforcement

$\phi_b$ : member reduction factor, generally 0.77

(3) In case where torsion and shear act conjointly, design shall be performed for the resultant assuming an equal inclination angle of the compression strut in ultra high

## Commentary

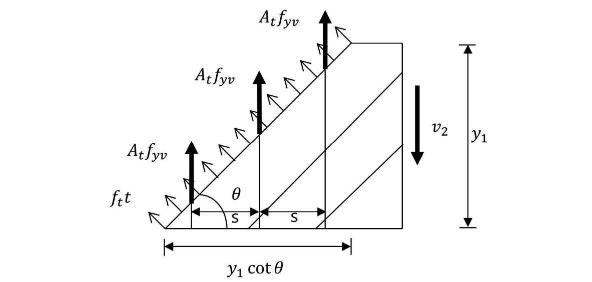



Fig. c.7.3.3 Shear force of second surface of equivalent thin–walled tube

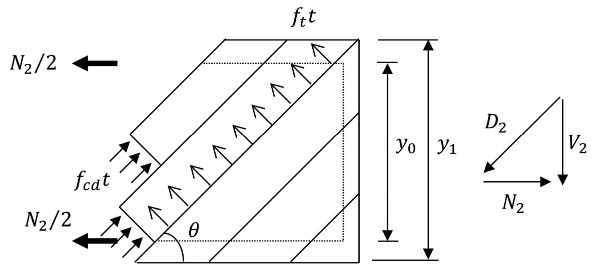



Fig. c.7.3.4 Longitudinal force of second surface of equivalent thin–walled tube

Figure c.7.3.5 compares the predicted torsional moments resulting from Equation (7.3.4) by applying the actual tensile strength obtained through tensile test instead of the design tensile strength to the experimental results obtained from the afore-mentioned 17 K-UHPC beams. It can be seen that Equation (7.3.4) computes rationally the ultimate torsional moment of the ultra high performance concrete beam.

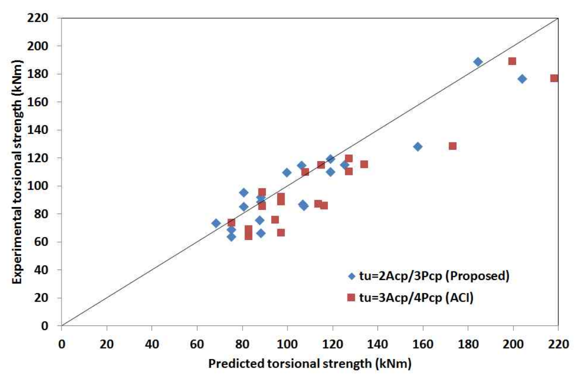



Fig. c.7.3.5 Comparison of ultimate torsional moment of K–UHPC beam

(3) Unlike ordinary concrete, ultra high performance concrete continues to transfer tension even after cracking. Therefore, this feature is considered for the computation of

**Specifications**

performance concrete. Even if the stress due to shear occurs over the whole width of the cross-section, the stress due to torsion shall satisfy Equation (7.3.6) obtained by assuming that it is resisted by the thin-walled tube.

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{9T_u p_h}{8A_{cp}^2}\right)^2} \le \frac{V_{yd}}{b_w d} \qquad (7.3.6)$$

(4) In case of hollow sections, Equation (7.3.7) shall be satisfied for the resultant of the shear stress caused by the torsion acting on the same section and the shear stress caused by the shear force.

$$\left(\frac{V_u}{b_w d}\right) + \left(\frac{9T_u p_h}{8A_{cp}^2}\right) \le \frac{V_{yd}}{b_w d} \qquad (7.3.7)$$

(5) If the wall thickness of the hollow section varies, calculation shall be at the position where the left-hand side of Equation (7.3.7) becomes maximum.

(6) If the actual wall thickness of the hollow

**Commentary**

the torsional strength amd shear strength. However, since the torsion-induced shear stress and the shear force-induced shear stress can be seen as acting simultaneously on the thin-wall in the case of hollow sections, design shall be performed for the resultant considering their directions. In the case of plain sections, the thin-walled tube theory assumes that the shear stress caused by torsion is resisted by the equivalent thin-wall at the outward of the section but the shear stress caused by shear force is resisted by the whole section. Therefore, these stresses as dealt in the form of sum of squares. In the case of ordinary concrete structure, the maximum shear force resisted by the stirrup is limited to prevent the occurrence of excessive cracks generated by the application of shear, torsion and resultant up to the service load level. However, the ultra high performance concrete structure does practically not experience cracking at the service load level owing to the action of the steel fibers and, even if cracking occurs, the developed crack width remains below 0.3 mm. Following, no limitation is provided for the maximum shear force resisted by the stirrup considering this feature.

## Specifications

section is smaller than $t = 2A_{cp}/3P_{cp}$, the second term in the left-hand side of Equation (7.3.7) shall be replaced by $\left(\frac{T_u}{2A_0^{real}t_{real}}\right)$ applying the actual wall thickness, where $A_0^{real}$ is the cross-sectional area delimited by the centerline of the shear flow acting on $t_{real}$.

## Commentary

Chapter 8

# Development and Splice of Reinforcement

Specifications

## 8.1 Nomenclature

$l_d =$ anchorage (development) length (mm)

$l_{db} =$ basic anchorage length (mm)

$d_b =$ nominal diameter of reinforcing bar, steel wire or tendon (mm)

$f_{yk} =$ characteristic yield strength of reinforcing bar (MPa)

$f_{ck} =$ characteristic compressive strength of K-UHPC (MPa)

$\phi_m =$ material reduction factor

## 8.2 Reinforcement Development Length

(1) The basic development length $l_{db}$ is determined by Equation (8.2.1) for the computation of the development length of the reinforcing bar (rebar) of structures using K-UHPC. Here, the minimum value of $l_{db}$ shall be larger than 5.5 times the diameter of the reinforcing bar. In case of shorter development length, test shall be performed to verify the satisfactory filling during placing of K-UHPC, and the durability and safety of the structure.

$$l_{db} = \frac{0.186 d_b f_{yk}}{\phi_m \sqrt{f_{ck}}} \qquad (8.2.1)$$

where

$l_{db}$ : basic development length

Commentary

## 8.1 Nomenclature

## 8.2 Reinforcement Development Length

(1) The current "Structural Concrete Design Code" does not specify regulations applicable to K-UHPC. The computation formula for the basic development length of ordinary concrete proposed in the "Structural Concrete Design Code" deals with the tension rebar and is expressed in Equation (c.8.2.1).

$$l_{db} = \frac{0.6 d_b f_y}{\sqrt{f_{ck}}} \qquad (c.8.2.1)$$

where

$l_{db}$ : basic development length

$d_b$ : nominal diameter of reinforcing bar, steel wire or tendon

$f_y$ : characteristic yield strength of reinforcing bar (MPa)

## Specifications

$d_b$ : nominal diameter of reinforcing bar, steel wire or tendon

$f_{yk}$ : characteristic yield strength of reinforcing bar

$f_{ck}$ : characteristic compressive strength of K-UHPC

$\phi_m$ : material reduction factor, generally 0.77

The development length $l_d$ is calculated by multiplying the basic development length $l_{db}$ by the correction factor of Table 8.2.1.

In addition, the term $\alpha$ used in Table 8.2.1 can be obtained as the reinforcement location factor as follows.

① Top reinforcement (horizontal reinforcement on which fresh concrete is placed by more than 300 mm below the development length or lap splice): $\alpha = 1.3$

② Other reinforcement: $\alpha = 1.0$

Table 8.2.1 Correction factor

| | Rebar < D19 | Rebar > D22 |
|---|---|---|
| In case where stirrup or tie reinforcement is arranged in quantities larger than the minimum amount of reinforcement all over $l_d$ while the clear spacing of the developed or spliced reinforcement is larger than $d_b$ and, the cover thickness is larger than $d_b$. Or, in case where the clear spacing of the developed or spliced reinforcement is larger than $2d_b$ and, the cover thickness is larger than $d_b$. | $0.8\alpha$ | $\alpha$ |
| Other | $1.2\alpha$ | $1.5\alpha$ |

## Commentary

$f_{ck}$ : characteristic compressive strength of concrete (MPa)

Equation (c.8.2.1) limits the effect of the compressive strength of concrete to 70 MPa, which makes it difficult to reflect the anchorage and bond performance of K-UHPC. The application of the currently specified Equation (c.8.2.1) as it is for the design of a structure using K-UHPC leads unavoidably to the need of securing excessive development length as shown in Figure c.8.2.1. Accordingly, the design Equation (8.2.1) was derived based upon test results evaluating the bond performance of K-UHPC.

According to the test results, sufficient bond performance is secured even with a development length 2.2 times the diameter of the reinforcement in case of reinforcement with 400 MPa class yield strength. Such bond performance is outstanding despite the development length reaches merely 1/8 of that recommended by the previous specified design Equation (c.8.2.1). However, the design Equation (8.2.1) was proposed by applying a sufficient safety margin of 250% to consider the convenience of construction or the construction error. Based upon Equation (8.2.1), the minimum development length shall be 5.5 times the diameter in case of deformed reinforcement with yield strength of 400 MPa for the 180 MPa-class K-UHPC.

**Specifications** | **Commentary**

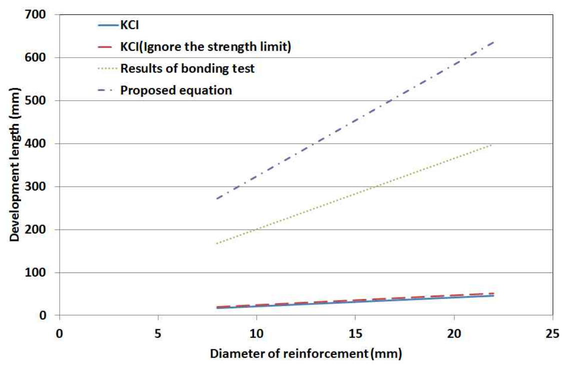



Fig. c.8.2.1 Development length of K-UHPC according to the diameter of reinforcement

## 8.3 Reinforcement Splicing

(1) The splice of the deformed reinforcement subjected to tension in the structure using K-UHPC shall comply with the "Structural Concrete Design Code". However, the development length $l_d$ of the tensile deformed reinforcement shall be calculated with respect to the method specified in section 8.2, and the minimum lap splice length shall be 100 mm.

(2) The splice of the deformed reinforcement subjected to compression shall comply with the "Structural Concrete Design Code". However, the lap splice length shall be 1/3 of that determined with respect to the "Structural Concrete Design Code", and the minimum lap splice length shall be 100 mm.

## 8.3 Reinforcement Splicing

(1) The determination of the lap splice length in the splice of the deformed reinforcement is governed by the effect of the development length. As explained in Commentary 8.2, K-UHPC exhibits outstanding bond performance with the deformed reinforcement compared to ordinary concrete. Even if sufficient safety margin is considered, the development length can be determined to about 1/3 of that of ordinary concrete. Following, the lap splice length is also set to a value corresponding to 1/3 of the lap splice length required in ordinary concrete.

Chapter 9

# Prestressed K-UHPC

**Specifications**

## 9.1 Scope of Application

(1) The specifications of this chapter apply in case where prestress is introduced in K-UHPC members using tendons like steel wire, steel bar and steel wire strand.

## 9.2 Design Assumptions

(1) The strength design of the prestressed K-UHPC member subjected to bending moment and axial force is similar to the strength design of the K-UHPC without tendon but shall consider additionally the contribution of the tendon.

(2) The allowable stress design of the prestressed K-UHPC member subjected to bending moment and axial force shall be performed under the assumption of linear elastic behavior.

## 9.3 Allowable Stress Design of Flexural Member

(1) In the allowable stress design of the prestressed K-UHPC member subjected to bending moment and axial force, the allowable stress of K-UHPC shall follow the specifications of Chapter 4.

(2) Serviceability shall be examined by

**Commentary**

## 9.1 Scope of Application

## 9.2 Design Assumptions

(1) The strength design of prestressed K-UHPC shall be performed basically based upon the assumptions of Chapter 6.

(2) This assumption is identical to that of ordinary prestressed concrete.

## 9.3 Allowable Stress Design of Flexural Member

(1), (2) During the calculation of the allowable stress of concrete, the applied strength of concrete shall be the strength at the time of check. However, the allowable stresses of concrete during tensioning and service can be seen as almost identical since the strength is

**Specifications**

comparing the allowable stress of concrete to the concrete stress prior to the loss of prestress with time immediately after the introduction of prestress, and the concrete stress caused by the service load after occurrence of loss of prestress.
(3) The allowable stress of the tendon shall follow the "Structural Concrete Design Code".

## 9.4 Loss of Prestress

(1) The following factors inducing the loss of prestress shall be considered for the determination of the effective prestress.
① Activity of the anchoring device
② Elastic shrinkage of concrete
③ Friction between the post-tension tendon and the duct
④ Creep of concrete
⑤ Drying shrinkage of concrete
⑥ Stress relaxation of tendon
(2) The characteristics of K-UHPC shall be considered during the calculation of the loss of prestress caused by creep and drying shrinkage.

## 9.5 Strength Design of Flexural Member

(1) During the assumption of the member failure, the stress of the tendon shall be calculated based upon the stress-strain curve of the tendon and the strain compatibility condition.

**Commentary**

nearly constant for K-UHPC after the execution of high temperature wet curing as mentioned in Chapter 2 and, considering the fact that tensioning is in most cases conducted after high temperature wet curing.

## 9.4 Loss of Prestress

(1) ①, ② and ③ are losses occurring immediately at the introduction of prestress, ④, ⑤ and ⑥ are losses occurring in a long-term.

(2) Long-term behaviors like the creep and drying shrinkage of K-UHPC differ from those of ordinary concrete. Therefore, the characteristics of K-UHPC presented in Chapter 2 shall be considered during the calculation of the loss of prestress due to long-term behaviors.

## 9.5 Strength Design of Flexural Member

(1) During the member failure proposed in the "Structural Concrete Design Code", it is recommended to avoid the use of the approximation equation of the tendon stress due to the difference in the behavior of ordinary concrete and K-UHPC. Accordingly, the tendon stress shall be obtained using the exact equation based upon the strain compatibility equation in the case of bonded tendon. However, since the strain compatibility cannot be established in the case of

**Specifications** | **Commentary**

unbonded tendon, the tendon stress during the member failure shall be calculated based on another method or test.

## 9.6 Design of Anchorage

(1) The design method of the anchorage zone can be basically applied as proposed by the "Structural Concrete Design Code". However, the effect of the design tensile strength $f_{td}$ of K-UHPC shall be additionally considered.

(2) The anchorage of the post-tensioned member can be achieved using the anchorage used in previous ordinary concrete.

## 9.6 Design of Anchorage

(1) Even if the tensile strength of concrete is neglected in the design of the anchorage zone of ordinary concrete, K-UHPC provides a remarkably larger design tensile strength $f_{td}$ compared to ordinary concrete. Therefore, the tensile strength of K-UHPC is considered so as to improve the efficiency during the design of the anchorage zone.

For example, when the strut-tie model is applied during the design of the anchorage zone, the tension of the tie in ordinary concrete is supported by the reinforcement or the tendon and the tensile strength of concrete is neglected. However, the tension of the tie in K-UHPC can be seen as being supported not only by the reinforcement or the tendon but also by concrete.

Besides, research results on K-UHPC revealed that the stiffening reinforcement used in the anchorage zone of ordinary concrete can be omitted for K-UHPC owing to its high tensile strength. However, the stiffening reinforcement can be applied to achieve conservative design.

(2) In order to take advantage of K-UHPC enabling to reduce significantly the cross sectional dimensions compared to ordinary concrete, it is desirable to use the optimized anchorage for K-UHPC. However, the use of previous anchorages is allowed considering the economic efficiency and workability.

Chapter 10

# Precast K-UHPC

**Specifications**

## 10.1 General

(1) This chapter specifies the requirements for structures made completely of K-UHPC precast members.

### 10.1.1 Nomenclature

$A_{cc}$ = compressive zone area of the whole cross section ($mm^2$)

$A_k$ = cross-sectional area of shear key in the compressive zone the whole cross section ($mm^2$)

$f_{ck}$ = characteristic compressive strength of K-UHPC (MPa)

$f_{nd}$ = average compressive stress acting perpendicularly to the shear plane (MPa)

$V_{cwd}$ = design shear strength for K-UHPC connection (N)

$V_k$ = shear strength contributed by shear key, (N), $V_k = 0.1 A_k f_{cd}$

$\beta$ = surface shape factor, generally between 0 and 1, to be determined through test according to the number of shear key and eventual presence of adhesive

**Commentary**

## 10.1 General

(1) The precast K-UHPC member is an element fabricated in a place that is not the construction site of the structure or fabricated in factory. The items relative to the general design of each member element are handled in the relevant specifications and not in this chapter.

### 10.1.1 Nomenclature

**Specifications**

(when adhesive is used in the joint: $\beta = 0.5$)

$\mu =$ coefficient of friction at the contact surface, generally 0.45

## 10.2 Design Principle

(1) Design check shall be performed considering the following circumstances during construction when designing the precast K-UHPC structural member.

① Removal of form
② Transport to the storehouse
③ Short and long-term storage (support and load conditions)
④ Transport to site
⑤ Positioning
⑥ Assembling and joints

(2) Special attention shall be paid on the following precast member details when designing the precast K-UHPC structural member.

① Fabrication process (dynamic effect)
② Temporary or permanent bearing
③ Connection and joints between elements

## 10.3 Precast K-UHPC Bridge

(1) The design of the precast K-UHPC girder shall examine all the load, restraint and unstable conditions that will be faced from the initial fabrication stage including the removal of form, storage, transport and erection to the completion stage of the superstructure.

**Commentary**

## 10.2 Design Principle

(1) The circumstances during construction of the precast K-UHPC structure correspond to important items in the design. Especially, the pre-tensioned precast member may experience bending moment during construction opposite to that encountered in service. Therefore, attention shall be paid during design.

(2) Dynamic effects occur often during transport and the erection process like the positioning. In such case, the dynamic effect can be considered by multiplying the static effect by an appropriate factor when precise dynamic analysis cannot be performed. According to the circumstances, this factor can take values between 0.8 and 1.2.

## 10.3 Precast K-UHPC Bridge

(1) Temporary supports or protection devices are necessary to secure the stability of the slender and long K-UHPC member during erection. The stiffness of such erection equipment is as much important as the strength of the member. The construction contractor is responsible for the safe storage, handling and erection of the precast member and, for securing appropriate equipment and methods for temporary support. In case where there are limitations for the position or dimensions of the temporary supports or lateral braces as per design, these limitations

**Specifications**

(2)The maximum dimensions and weight of the precast member fabricated in a place other than the site shall comply with the transport limitation specified regionally.

(3) In case where the surface of the member equipped with an anchorage for the lifting device it is predicted to be exposed to the vision of pedestrian or exposed to corrosive substances after completion, the burying emplacement of the lifting device, the removal depth and the method for filling this empty space shall be explicitly mentioned in the construction contractual documents.

## 10.4 Requirements of Precast Connection

(1) The materials used in the connection shall satisfy the following conditions.

① The materials shall secure stability and durability all along the life of the structure.

② The materials shall be compatible chemically and physically.

③ The materials shall be protected against unfavorable chemical and physical actions.

④ The materials shall exhibit fire resistance agreeing with the fire resistance of the structure.

(2) The connection shall be able to secure sufficient strength and stiffness structurally required for the load effect agreeing with the design conditions.

(3) The compressive strength of the precast K-UHPC segment shall be larger than 17 MPa before the removal of form.

(4) The joint interface between the segments shall be intentionally made rough so as to expose the aggregates or shall be installed with shear keys. Here, the treatment method of the joint interface shall be mentioned in

**Commentary**

shall be explicitly mentioned in the contractual documents.

(2) The size and weight limits of the goods permitted for road transport are important. For long members, survey shall be conducted in advance to guarantee the transportability prior to design.

(3) The contractor can select the form of the lifting device of the precast member and is responsible for the performances of the selected device. In concern with the anchorage of the lifting device, the anchoring device can be a loop made of PS steel wire or reinforcing bar of which ends are embedded in concrete or a spiral bar directly embedded in concrete.

## 10.4 Requirements of Precast Connection

(4) In order to increase the friction of the joint interface, it is necessary to install shear keys in the vertical joint surface. In the absence of shear key, particular attention shall be paid in forming a rough joint interface for

**Specifications**

the design drawings. In case where tendons are crossed, the width of the cast-in-place bonded joint shall be sufficiently wide to allow the connection of the ducts.

(5) The connection between each segment can be achieved using cast-in-place bonded joint and epoxy adhesive joint. For the epoxy adhesive joint, prestress shall be introduced by applying a compressive stress larger than minimum 0.2 MPa all over the jointed surface until the hardening of epoxy.

## 10.5 Shear Strength of Joint

(1) The shear strength of the joint shall be larger than the developed shear force. The design shear strength $V_{cwd}$ for the joint can be obtained as follows considering the shear contribution of the shear keys.

$$V_{cwd} = \phi_b(\mu f_{cd}{}^{\beta} f_{nd}{}^{(1-\beta)} A_{cc} + V_k) \qquad (10.5.1)$$

where

$\mu$ : coefficient of friction at the contact surface, generally 0.45

$f_{cd}$ : design compressive strength of K-UHPC

$\beta$ : surface shape factor, generally between

**Commentary**

the bonding of the segments so as to secure the prescribed shear strength. The web shear key of the precast K-UHPC member shall as possible be installed widely and extensively in the web cross-section. The details of the web shear key can refer to Figure c.10.4.1. Moreover, the shear key can also be installed in the top and bottom flanges. Slightly larger single shear keys can be used in the top and bottom flanges.

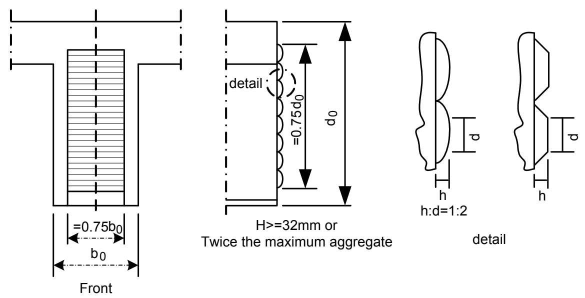



Fig. c.10.4.1 Examples of shear key

(5) A prescribed temporary compressive stress is necessary to secure perfect bonding and prevent irregular epoxy thickness.

## 10.5 Shear Strength of Joint

(1) Since the joint using shear key is designed to prevent the occurrence of tensile stress, sufficient frictional resistance can be expected from the prestress force. Accordingly, the check for serviceability can be skipped under the condition that check for safety is performed.

**Specifications**

0 and 1, to be determined through test according to the number of shear key and eventual presence of adhesive (when adhesive is used in the joint: $\beta$ = 0.5)

$f_{nd}$ : average compressive stress acting perpendicularly to the shear plane

$A_{cc}$ : compressive zone area of the whole cross section

$V_k$ : contribution to shear strength by shear key, $V_k = 0.1 A_k f_{cd}$

$A_k$ : cross-sectional area of shear key in the compressive zone the whole cross section

$\phi_b$ : member reduction factor, generally 0.77

(2) Safety check shall be performed for the bearing stress developed in the shear keys.

**Commentary**

## Chapter 1 General Provisions

1.1 Korea Road & Transportation Association, "Highway Bridge Design Code", Korea Ministry of Land, Transport and Maritime Affairs, 2010. (in Korean)

1.2 Korea Concrete Institute, "Structural Concrete Design Code", Korea Ministry of Land, Transport and Maritime Affairs, 2012. (in Korean)

## Chapter 2 Design Material Properties

2.1 Korea Institute of Construction Technology, "Development of the advanced technology of toughness in ultra high performance concrete for hybrid cable stayed bridge", Report of 6th Year of Super Bridge 200, 2012(to be published). (in Korean)

2.2 Korea Institute of Construction Technology, "Development of ultra high performance concrete for hybrid stayed cable bridge", Report of 6th Year of Super Bridge 200, 2012(to be published). (in Korean)

2.3 Ryu, G.S., Kim, S.H., Ahn, G.H., Koh, K.T., "Evaluation of the direct tensile behavior characteristics of UHPC using twisted steel fibers", Advanced Materials Research, Vols. 602-604, 2012, pp. 96-101.

2.4 Kim, S.H., Ryu, G.S., Koh, K.T., "Flexural behavior characteristics of UHPC according to twisted steel fiber", Advanced Materials Research, Vols. 652-654, 2013, pp. 1499-1504.

2.5 Nguyen D.L., Kim, D.J., Ryu, G.S., Koh, K.T., "Size effect on flexural behavior of ultra-high-performance hybrid fiber-reinforced concrete", Composites: Part B, Vol. 45, 2013, pp. 1104-1116.

2.6 Park, S.H., Kim, D.J., Ryu, G.S., Koh, K.T., "Tensile Behavior of Ultra High Performance Hybrid Fiber Reinforced Concrete", Cement & Concrete Composites, No. 34(2), 2012, pp. 172-184.

2.7 Yoo, D.Y., Park, J.J., Kim, S.W., Yoon, Y.S., "Properties and Prediction Model for Ultra High Performance Fiber Reinforced Concrete (UHPFRC): (I) Evaluation of Setting and Shrinkage Characteristics and Tensile Behavior", Journal of KSCE, Vol. 32, No. 5A, 2012, pp. 307-315. (in Korean)

2.8 Yoo, D.Y., Park, J.J., Kim, S.W., Yoon, Y.S., "Properties and Prediction Model for Ultra High Performance Fiber Reinforced Concrete (UHPFRC): (II) Evaluation of Restrained Shrinkage Characteristics and Prediction of Degree of Restraint", Journal of KSCE, Vol. 32, No. 5A, 2012, pp. 317-325. (in Korean)

2.9 Kim, D.J., Park, S.W., Ryu, G.S., Koh, K.T., "Comparative flexural behavior of Hybrid Ultra High Performance Fiber Reinforced Concrete with different macro fibers", Construction and Building Materials, Vol. 25, 2011, pp. 4144-4155.

2.10 Choi, S.Y., Joh, C., Cho, J.R., Park, J., "Implementation of a Thermo-Chemo-Mechanical Model of Early Age UHPC", KSCE Journal of Civil Engineering, Vol. 16(7), 2012, pp. 1218-1224.

2.11 Korea Institute of Construction Technology, "Development of the advanced technology of toughness in ultra high performance concrete for hybrid cable stayed bridge", Report of 5th Year of Super Bridge 200, KICT 2011-127, 2011. (in Korean)

2.12 Korea Institute of Construction Technology, "Development of ultra high performance concrete for hybrid stayed cable bridge", Report of 5th Year of Super Bridge 200, KICT 2011-081, 2011. (in Korean)

2.13 Yoo, D.Y., Park, J.J., Kim, S.W., Yoon, Y.S., "Characteristics of Early-Age Restrained Shrinkage and Tensile Creep of Ultra-High Performance Cementitious Composites (UHPCC)", Journal of the Korea Concrete Institute, Vol. 23, No. 5, 2011, pp. 581-590. (in Korean)

2.14 Yoo, D.Y., Park, J.J., Kim, S.W., Yoon, Y.S., "Evaluating Early Age Shrinkage Behavior of Ultra High Performance Cementitious Composites (UHPCC) with CSA Expansive Admixture and Shrinkage Reducing Agent", Journal of the Korea Concrete Institute, Vol. 23, No. 4, 2011, pp. 441-448. (in Korean)

2.15 Ryu, G.S., Kang, S.T., Park, J.J., Koh, K.T., Kim, S.W., "Mechanical Behavior of UHPC(Ultra High Performance Concrete) according to Hybrid Use of Steel Fibers", Advanced Materials Research, Vols. 287-290, 2011, pp. 453-457.

2.16 Kim, S.W., Choi, S., Lee, K.M., Park, J.J., "Autogeneous Shrinkage Characteristics of Ultra High Performance Concrete", Journal of the Korea Concrete Institute, Vol. 23, No. 3, 2011, pp. 295-301. (in Korean)

2.17 Koh, K.T., Ryu, G.S., Kang, S.T., Park, J.J., Kim, S.W., "Shrinkage Properties of Ultra-High Performance Concrete (UHPC)", Advanced Science Letters, Vol. 4, 2011, pp. 948-952.

2.18 Yoo, D.Y., Park, J.J., Kim, S.W., Yoon, Y.S., "Combined influence of expansive and shrinkage reducing admixtures on shrinkage characteristics of ultra-high performance fiber reinforced concrete (UHPFRC)", Cement and Concrete Composites, 2011.

2.19 Hong, K.N., Kang, S.T., Kim, S.W., Park, J.J., Han, S.H., "Material properties of air-cured ultra-high-performance steel-fiber-reinforced concrete at early ages", International Journal of the Physical Sciences, Vol. 5(7), 2011, pp. 2622-2634.

2.20 Korea Institute of Construction Technology, "Development of the advanced technology of toughness in ultra high performance concrete for hybrid cable stayed bridge", Report of 4th Year of Super Bridge 200, KICT 2010-127, 2010. (in Korean)

2.21 Korea Institute of Construction Technology, "Development of ultra high performance concrete for hybrid cable stayed bridges(manual of manufacturing UHPC)", Report of 4th Year of Super Bridge 200, KICT 2010-114, 2010. (in Korean)

2.22 Kang, S.T., Park, J.J., Ryu, G.S., Kim, S.W., "The Effect of Steel-Fiber Reinforcement on the Compressive Strength of Ultra High Performance Cementitious Composites(UHPCC)", Journal of the Korea Institute for Structural Maintenance and Inspection, Vol. 14. No. 5, 2010, pp. 110-118. (in Korean)

2.23 Cha, S.W., Kim, K.H., Kim, S.W., Park, J.J., Bae, S.G., "Models for Hydration Heat Development and Mechanical Properties of Ultra High Performance Concrete", Journal of the Korea Concrete Institute, Vol. 22, No. 3, 2010, pp. 389-397. (in Korean)

2.24 Hong, K.N., Kang, S.T., Kim, S.W., Park, J.J., Han, S.H., "Mechanical Properties of an Air Cured Ultra High Performance Steel Fiber Reinforced Concrete at Early Age", Cement and Concrete Research, 2010.

2.25 Korea Institute of Construction Technology, "Development of Ultra High Performance Concrete for Hybrid Cable Stayed Bridge (Improving Construction Works)", Report of 3rd Year of Super Bridge 200, KICT 2009-076, 2009. (in Korean)

2.26 Korea Institute of Construction Technology, "Development of the advanced technology of toughness in ultra high performance concrete for hybrid cable stayed bridge", Report of 3rd Year of Super Bridge 200, KICT 2009-077, 2009. (in Korean)

2.27 Kwak, H.G., Na, C., Kim, S.W., Kang, S.T., "Tension-Stiffening Model and Application of Ultra High

Strength Fiber Reinforced Concrete", Journal of KSCE, Vol. 29, No. 4A, 2009, pp. 267-279. (in Korean)

2.28 Kang, S.T., Hong, K.N., Han, S.H., Kim, S.W., "The Effect of Fiber Volume Fraction on the Tension Softening Behavior of Steel Fiber-Reinforced Ultra High Strength Concrete", Journal of the Korea Concrete Institute, Vol. 21, No. 1, 2009, pp. 13-20. (in Korean)

2.29 Korea Institute of Construction Technology, "Development of ultra high performance concrete for hybrid stayed cable bridge", Report of 2nd Year of Super Bridge 200, KICT 2008-073, 2008. (in Korean)

2.30 Kang, S.T., Park, J.J., Ryu, G.S., Kim, S.W., "Influence of the Filler"s Particle Size on the Mechanical Properties of Ultra High Performance Concrete(UHPC)", Journal of KSCE, Vol. 28, No. 4A, 2008, pp. 573-580. (in Korean)

2.31 Korea Institute of Construction Technology, "Development of ultra high performance concrete for hybrid stayed cable bridge", Report of 1st Year of Super Bridge 200, KICT 2007-104, 2007. (in Korean)

2.32 Koh, K.T., Park, J.J., Ryu, G.S., Kang, S.T., "Effect of the Compressive Strength of Ultra-High Strength Steel Fiber Reinforced Cementitious Composites on Curing Method", Journal of KSCE, Vol. 27, No. 3A, 2007, pp. 427-432. (in Korean)

2.33 KS F 2438, "Testing Method for Static Modulus of Elasticity and Poisson's ratio in Compression of Cylindrical Concrete Specimens", Korea Standards Association, 2007. (in Korean)

2.34 FHWA, "Material Property Characterization of Ultra-High Performance Concrete", Publication No. FHWA-HRT-06-103, FHWA, U.S., 2006.

2.35 JSCE, "Provisional Recommendations for the Design and Construction of Ultra High Strength Fiber-Reinforced Concrete ", JSCE, Japan, 2004. (in Japanese)

2.36 AFGC-SETRA, "Ultra High Performance Fiber-Reinforced Concretes Interim Recommendations", Bagneux, France, 2002.

## Chapter 3 Design Loads and Load Combinations

3.1 Korea Rail Network Authority, "Railway Design Code(Roadbed)", Korea Ministry of Land, Transport and Maritime Affairs, 2011. (in Korean)

3.2 Korea Road & Transportation Association, "Highway Bridge Design Code", Korea Ministry of Land, Transport and Maritime Affairs, 2010. (in Korean)

3.3 Korea Concrete Institute, "Structural Concrete Design Code", Korea Ministry of Land, Transport and Maritime Affairs, 2012. (in Korean)

3.4 Korea Concrete Institute, "Provisional Design Recommendations for Concrete Cable Stayed Bridge", Korea Concrete Institute, 2008. (in Korean)

3.5 Korean Society of Civil Engineers, "Provisional Design Recommendations for Cable-Supported Bridge", Korean Society of Civil Engineers, 2005. (in Korean)

## Chapter 4 Serviceability and Durability

4.1 Kwahk, I., Lee, J., Kim, J., Joh, C, "Evaluation of the Crack Width of the Ultra High Performance Concrete(K-UHPC) Structures", Journal of the Korean Society of Safety, Vol. 27, No. 6, 2012, pp. 99-108. (in Korean)

4.2 Korea Institute of Construction Technology, "Design Specific Technology for Ultra Performance Concrete", Report of 5th Year of Super Bridge 200, KICT 2011-078, 2011. (in Korean)
4.3 Kook, K.H., Shin, H.O., Kwahk, I.J., Yoon, Y.S., "Bond Characteristics of Ultra-High-Performance Concrete", Journal of the Korea Concrete Institute, Vol. 22, No. 6, 2010, pp. 753-760. (in Korean)
4.4 Kang, S.T., "Flow-dependent fiber orientation distribution and its effect on the tensile behavior of ultra high performance cementitious composites", Doctoral Thesis, Dept. of Civil and Environmental Engineering, KAIST, 2010, pp. 68-71. (in Korean)
4.5 Deutscher Ausschuss für Stahlbeton (DafStb): "Sachstandsbericht Ultrahochfester Beton", 1. Auflage, Beuth Verlag GmbH, Berlin, 2008, pp. 36-38.
4.6 Fehling, E., Leutbecher, T., "Tensile behavior of ultra high performance concrete(UHPC) reinforced with a combination of steel-fibers and rebars", 3rd International Conference on Structural Engineering Mechanics and Computation(SEMC), Cape Town, Proc., 2007, pp. 549-550.
4.7 Park, J.J., Koh, K.T., Ryu, G.S., Kim, S.W., "Evaluation on Durability of Ultra-High Strength Cementitious Composites", Journal of KSCE, Vol. 27, No. 2A, 2007, pp. 257-263. (in Korean)
4.8 Jungwirth, J., "Zum Tragverhalten von Zugbeanspruchten Bauteilen aus Ultra-Hochleistungs-Faserbeton", Doctoral Thesis No. 3429, Faculté Environnement Naturel, Architectural et Construit, Ecole Polytechnique Fédérale de Lausanne, 2006, pp. 15-17.
4.9 Korea Institute of Construction Technology, "Development of Technology to Improve the Durability of Concrete Bridges", Final Report of Super Bridge 200, KICT 2006-089, 2006, pp. 290-314. (in Korean)
4.10 Fehling, E., Schmidt, M., Teichmann, Th., Bunje, K., Bornemann, R. und Middendorf, B., "Entwicklung, Dauerhaftigkeit und Berechnung Ultra-Hochfester Betone(UHPC)", Forschungsbericht DFG FE 497/1-1, Schriftenreihe Baustoffe und Massivbau, Heft 1, Universität Kassel, 2005, pp. 123.
4.11 Kang, B.S., "Calculation of Crack Width in SFRC Structures", Journal of the Korean Society for Railway, Vol. 8, No. 4, 2005, pp. 293–298. (in Korean)
4.12 JSCE, "Provisional Recommendations for the Design and Construction of Ultra High Strength Fiber-Reinforced Concrete ", JSCE, Japan, 2004. (in Japanese)
4.13 Pyfl, Th., "Tragverhalten von Stahlfaserbeton", Dissertation ETH Nr. 15005, ETH Zurich, 2003, pp. 27-29.
4.14 Ma, J., Schneider, H., and Wu, Z., "Bruchmechanische Kenngrößen von UHFB", Innovationen im Bauwesen, Ultrahochfester Beton (Hrsg.: König, G., Holschemacher, K. und Dehn, F.), Bauwerk Verlag, Berlin, 2003, pp. 121-130.
4.15 AFGC-SETRA, "Ultra High Performance Fiber-Reinforced Concretes Interim Recommendations", Bagneux, France, 2002.

## Chapter 5 Structure Details

5.1 Choi, S., Lim, B., Oh, C.K., Joh, C., "Technique for the Measurement of Crack Widths at Notched / Unnotched Regions and Local Strains", Journal of the Korea Concrete Institute, Vol. 25, No. 2, 2012, pp. 205-214. (in Korean)
5.2 Hwang, H.H., Yeo, I.S., Cho, K.H., Park, S.Y., "Evaluation of Flexural Strength for UHPC Deck Joints with Lap-Spliced Reinforced Steel Bar", Journal of the Korea Institute for Structural Maintenance and Inspection, Vol. 15, No. 6, 2011, pp. 92-99. (in Korean)
5.3 Yoo, D.M., Hwang, H.H., Kim, S.T., Park, S.Y., "An Experimental Study on the shear connection for

UHPC Deck Bridge", Journal of the Korean Society for Composite Materials, Vol. 24, No. 5, 2011, pp. 29-33. (in Korean)

5.4 Korea Institute of Construction Technology, "Design Specific Technology for Ultra Performance Concrete", Report of 4th Year of Super Bridge 200, KICT 2010-107, 2010. (in Korean)

5.5 Kook, K.H., Shin, H.O., Kwahk, I.J., Yoon, Y.S., "Bond Characteristics of Ultra-High-Performance Concrete", Journal of the Korea Concrete Institute, Vol. 22, No. 6, 2010, pp. 753-760. (in Korean)

5.6 Lee, G.Y., Kim, W., Lee, H.M., "Tension Lap Splice Length in High-Strength Concrete Flexural Members", Journal of the Korea Concrete Institute, Vol. 21, No. 6, 2009, pp. 753-761. (in Korean)

5.7 Jung, J.H., Kang, H., Ahn, J.M., Shin, S.W., "Bond Stress between Reinforcement and Ultra High-Strength Concrete with a Compressive Strength of 150MPa", Journal of Architectural Institute of Korea - Structure & Construction, Vol. 24, No. 12, 2008, pp. 43-50. (in Korean)

5.8 Deutscher Ausschuss für Stahlbeton (DafStb): "Sachstandsbericht Ultrahochfester Beton", 1. Auflage, Beuth Verlag GmbH, Berlin, 2008, pp. 36-38.

5.9 Korea Institute of Construction Technology, "Development of Integrated Technology for Structural System of Hybrid Cable Stayed Bridge", Report of 2nd Year of Super Bridge 200, KICT 2008-070, 2008. (in Korean)

5.10 JSCE, "Provisional Recommendations for the Design and Construction of Ultra High Strength Fiber-Reinforced Concrete ", JSCE, Japan, 2004. (in Japanese)

## Chapter 6 Bending and Compression

6.1 Yang, I.H., Joh, C., Kim, B.S., "Flexural Response Predictions for Ultra High Performance Fiber-Reinforced Concrete Beams", Magazine of Concrete Research, 64(2), 2012, pp. 113-127.

6.2 Yang, I.H., Joh, C., Kim, B.S., "Flexural Strength of Large Scale Ultra High Performance Concrete Prestressed T-Beams", Canadian Journal of Civil Engineering, 38, 2011, pp. 1185-1195.

6.3 Yang, I.H., Joh, C., Kim, B.S., "Moment-Curvature Analysis of Steel Fiber-Reinforced Ultra High Performance Concrete Beams with Tension Softening Behavior", Journal of Computational Structural Engineering Institute of Korea, Vol. 2, No. 3, 2011, pp. 237-248. (in Korean)

6.4 Kang, S.T., Ryu, G.S., Park, J.J., Koh, K.T., Kim, S.W., "Flexural Performance of Lightly Reinforced Concrete Beams with Ultra-High Strength Fiber-Reinforced Concrete (UHSFRC)", Advanced Science Letters, Vol. 4, No. 3, 2011, pp. 1032-1038.

6.5 Yang, I.H., Joh, C., Kim, B.S., "An Experimental Study on Flexural Behavior of Steel Fiber Reinforced Ultra High Performance Concrete Prestressed Girders", Journal of the Korea Concrete Institute, Vol. 22, No. 6, 2010, pp. 777-786. (in Korean)

6.6 Yang, I.H., Joh, C., Kim, B.S., "Structural behavior of ultra high performance concrete beams subjected to bending", Engineering Structures, 32, 2010, pp. 3478-3487.

6.7 Yang, I.H., Joh, C., "Prediction of Flexural Capacity of Steel Fiber-Reinforced Ultra High Strength Concrete Beams", Journal of KSCE, Vol. 30, No. 3A, 2010, pp. 317-328. (in Korean)

6.8 Korea Institute of Construction Technology, "Design Specific Technology for Ultra High Performance Concrete", Report of 3rd Year of Super Bridge 200, KICT 2009- 095, 2009. (in Korean)

6.9 Yang, I.H., Joh, C., Kang, S.T., Kim, B.S., "An Experimental Study on Flexural Behavior of Steel Fiber Reinforced Ultra High Performance Concrete", Journal of the Korea Concrete Institute, Vol. 21, No. 6, 2009, pp. 737-744. (in Korean)

6.10 JSCE, "Provisional Recommendations for the Design and Construction of Ultra High Strength Fiber-Reinforced Concrete ", JSCE, Japan, 2004. (in Japanese)

6.11 AFGC-SETRA, "Ultra High Performance Fiber-Reinforced Concretes Interim Recommendations", Bagneux, France, 2002.

## Chapter 7 Shear and Torsion

7.1 Korea Institute of Construction Technology, "Design Specific Technology for Ultra Performance Concrete", Report of 6th Year of Super Bridge 200, 2012(to be published).

7.2 Yang, I.H., Joh, C., Lee, J.W., Kim, B.S., "An Experimental Study on Shear Behavior of Steel Fiber-Reinforced Ultra High Performance Concrete Beams", Journal of KSCE, Vol. 32, No. 1A, 2012, pp. 55-64. (in Korean)

7.3 Yang, I.H., Joh, C., Kim, B.S., "Shear Behaviour of Ultra High Performance Fibre-Reinforced Concrete Beams without Stirrups", Magazine of Concrete Research, Vol. 64(11), 2012, pp. 979-993.

7.4 Joh, C., Kim, B.S., Hwang, H.H., Choi, K.K., Choi, S.H., "Punching Shear Strength of Deck Slabs Made of Ultra High Performance Concrete", Journal of the Korea Institute for Structural Maintenance and Inspection, Vol. 15, No. 4, 2011, pp. 221-231. (in Korean)

7.5 Korea Institute of Construction Technology, "Design Specific Technology for Ultra Performance Concrete", Report of 4th Year of Super Bridge 200, KICT 2010-107, 2010. (in Korean)

7.6 JSCE, "Provisional Recommendations for the Design and Construction of Ultra High Strength Fiber-Reinforced Concrete ", JSCE, Japan, 2004. (in Japanese)

7.7 AFGC-SETRA, "Ultra High Performance Fiber-Reinforced Concretes Interim Recommendations", Bagneux, France, 2002.

## Chapter 8 Anchorage and Splice

8.1 Kook, K.H., Shin, H.O., Kwahk, I.J., Yoon, Y.S., "Bond Characteristics of Ultra-High-Performance Concrete", Journal of the Korea Concrete Institute, Vol. 22, No. 6, 2010, pp. 753-760. (in Korean)

8.2 Lee, G.Y., Kim, W., Lee, H.M., "Tension Lap Splice Length in High-Strength Concrete Flexural Members", Journal of the Korea Concrete Institute, Vol. 21, No. 6, 2009, pp. 753-761. (in Korean)

8.3 Korea Institute of Construction Technology, "Design Specific Technology for Ultra Performance Concrete", Report of 3rd Year of Super Bridge 200, KICT 2009-095, 2009. (in Korean)

8.4 Jung, J.H., Kang, H., Ahn, J.M., Shin, S.W., "Bond Stress between Reinforcement and Ultra High-Strength Concrete with a Compressive Strength of 150MPa", Journal of Architectural Institute of Korea - Structure & Construction, Vol. 24, No. 12, 2008, pp. 43-50. (in Korean)

8.5 American Concrete Institute (ACI), "Building code requirements for structural concrete and commentary", ACI 318-11, Farmington Hill, Mich., 2011.

8.6 Hegge, J., Tuchlinski, D., and Kommer, B., "Bond Anchorage Behavior and Shear Capacity of Ultra High Performance Concrete Beams", Proceedings of the Second International Symposium on Ultra High Performance Concrete, Kassel, Germany, 2008, pp. 351-359.

8.7 Holschemacher, K., Weiße, D., and Klotz, S., "Bond of Reinforcement in Ultra High Strength

Concrete", Proceedings of International Symposium on Ultra High Performance Concrete, Kassel, Germany, 2004, pp. 375-388.

8.8 JSCE, "Provisional Recommendations for the Design and Construction of Ultra High Strength Fiber-Reinforced Concrete ", JSCE, Japan, 2004. (in Japanese)

8.9 Weiße, D. and Holschemacher, K., "Some Aspects about the Bond of Reinforcement in Ultra High Strength Concrete", Leipzig Annual Civil Engineering Report (LACER), No. 8, University of Leipzig, Germany, 2003, pp. 251-263.

8.10 Holschemacher, K., Weiße, D., and Dehn, F., "Influence of the Rebar Position on the Bond Behaviour in High Strength Concrete", Leipzig Annual Civil Engineering Report (LACER), No. 7. University of Leipzig, Germany, 2002, pp. 135-144.

8.11 AFGC-SETRA, "Ultra High Performance Fiber-Reinforced Concretes Interim Recommendations", Bagneux, France, 2002.

8.12 RILEM, "Technical Recommendations for the Testing and Use of Construction Materials: RC 6, Bond Test for Reinforcement Steel. 2. Pull-out Test", 1970.

## Chapter 9 Prestressed K-UHPC

9.1 Korea Institute of Construction Technology, "Design Specific Technology for Ultra Performance Concrete", Report of 4th Year of Super Bridge 200, KICT 2010-107, 2010. (in Korean)

9.2 Toutlemonde, F., Renaud, J.-C., Lauvin, L., Behloul, M., Simon, A. and Vildare, S., "Testing and analysing innovative design of UHPFRC anchor blocks for post-tensioning tendons", 6th International Conference on Fracture Mechanics of Concrete and Concrete Structures (FRAMCOS-6), Catania, Italy, 2007, pp. 17-22.

## Chapter 10 Precast K-UHPC

10.1 Lee, C.H., Kim, Y.J., Chin, W.J., Choi, E.S., "Experimental Study on the Shear Behavior of Ultra High Performance Precast PSC Bridge Joint with Joint Type and Lateral Force", Journal of KSCE, Vol. 31, No. 5A, 2011, pp. 379-387. (in Korean)

10.2 Lee, C.H., Chin, W.J., Choi, E.S., Kim, Y.J., "An Experimental Study on the Joints in Ultra High Performance Precast Concrete Segmental Bridges", Journal of the Korea Concrete Institute, Vol. 23, No. 2, 2011, pp. 235-244. (in Korean)

## 1.1 Scope of Application

This standard applies for the fabrication method of specimens purposed for compressive strength test, flexural strength test and direct tensile strength test of K-UHPC.

## 1.2 Quoted Standards

KS F 2401 Sampling Method for Fresh Concrete
KS F 2403 Method of Making Concrete Specimens for Strength Test
KS F 2425 Method of Making Test Sample of Concrete in the Laboratory

## 1.3 Specimen for Compressive Strength Test

(1) Dimensions of specimen
The specimen shall be a cylinder with height twice its diameter. This diameter shall be larger than 3 times the length of steel fiber or 100 mm. In general, the adopted diameter of the specimen is 100 mm, and the length is 200 mm.
(2) Mold
① The mold shall be made of an non-absorbable material that does not corrode in cement.
② The mold shall be free of deformation and water leakage when fabricating the specimen.
③ The inner surface of the mold shall be coated by a mineral grease or a form coating material before placing K-UHPC.
(3) Fabrication of specimen
① The mold shall be filled by placing slowly and continuously K-UHPC up to the upper face to prevent the occurrence of entrapped air.
② Since K-UHPC exhibits self-compacting property, no need is to use a tamping rod nor an internal vibrator. However, even if it is recommended to tap appropriately the mold using a rubber hammer to remove the air voids entrapped in K-UHPC, attention shall be paid to the possibility of the subsidence or the modification of the orientation of the steel fibers in case of excessive pounding.
(4) Finishing of upper face of specimen
The finishing of the upper surface of the specimen for compressive strength test of K-UHPC shall be performed in compliance to the grinding of KS F 2403.

## 1.4 Specimen for Flexural Strength Test

(1) Dimensions of specimen
The specimen shall be a prism with square cross-section of which the length of one side shall be longer than 4 times the length of the steel fibers or 100 mm. The length of the specimen shall be longer by 80 mm than 3 times the length of one side of the cross section. In general, specimens of 100×100×400 mm are used.
(2) Fabrication of specimen
The mold shall be disposed horizontally and placing shall be done slowly and continuously along the longitudinal direction up to the upper face so as to prevent the occurrence of entrapped air in

K-UHPC. The placing range shall be within L/4 (L: length of specimen) from the end of the mold considering the maximum bending moment region. If confluence takes place corresponding to the maximum bending moment region during placing, this region becomes a weak region. Therefore, placing shall be avoided at both ends of the mold. The compaction method is identical to the fabrication method of the compressive strength specimen in 1.3(3)②.

## 1.5 Specimen for Direct Tensile Strength Test

(1) Dimensions of specimen

① The unnotched specimen for direct tensile strength test of K-UHPC is shown in Figure A.1.5.1.

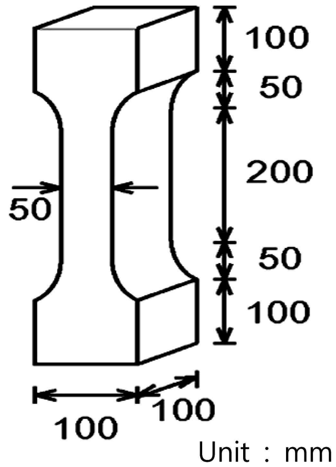



Fig. A.1.5.1 Unnotched specimen for direct tension

② The notched specimen for direct tensile strength test of K-UHPC is shown in Figure A.1.5.2. After completion of high temperature wet curing, the direct tension specimen is notched with a width of 2 mm and depth of 12.5 mm on both sides at its central part in order to induce cracking of the central part.

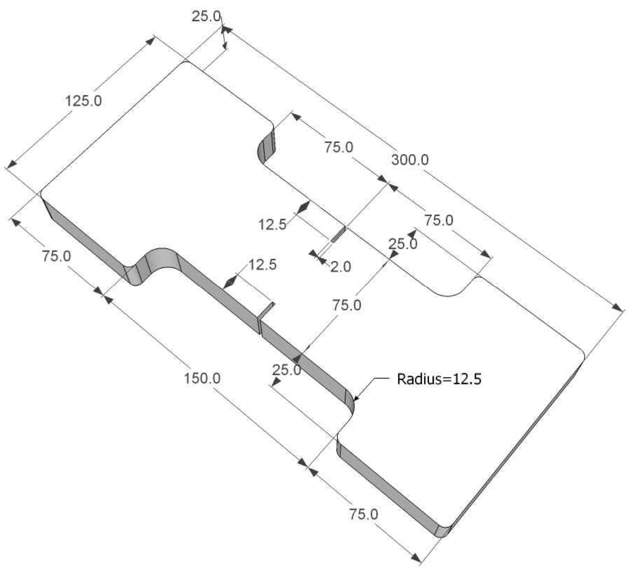



Fig. A.1.5.2 Notched specimen for direct tension (unit: mm)

(2) Fabrication of specimen

① Placing shall be done slowly and continuously from the end of the mold along the longitudinal direction up to the upper face so as to prevent the occurrence of entrapped air in K-UHPC. Attention shall be paid to prevent the occurrence of joint.

② The compaction method is identical to 1.3(3)②.

## 1.6 Stripping and Curing

(1) The placed surface shall be covered by a glass plate, steel plate or polyester film after placing of K-UHPC until stripping to prevent the evaporation of moisture.
(2) The curing of K-UHPC shall be identical to that of actual members. The curing process shall in principle proceed by early curing under wet condition at 20±2℃ during 24 hours after placing, followed by high temperature wet curing at 90±5℃ during 48 hours after stripping .

## 1.7 Report

Report shall be done for the following items.
(1) Designation of specimens
(2) Objective of test
(3) Temperature and humidity of laboratory
(4) Mix composition of K-UHPC
(5) Type and quality of adopted materials
(6) Type and capacity of mixer, amount of 1 mixing batch, mixing sequence and method
(7) Fabrication day and time of specimens
(8) Temperature, flow and air content of K-UHPC
(8) Curing method
(9) Appearance of specimens

## 2.1 Scope of Application

This standard applies for the compressive strength test method of K-UHPC specimens.

## 2.2 Quoted Standards

KS B 5533 Compression Testing Machines
KS F 2405 Standard Test Method for Compressive Strength of Concrete

## 2.3 Specimen

(1) The fabrication of the specimen shall comply with "Appendix 1: Fabrication Method of Specimens for Strength Test of K-UHPC".
(2) The diameter and height shall be measured up to 0.1 mm and 1 mm, respectively. The diameter shall be measured at mid-height of the specimen along two orthogonal directions.
(3) In presence of loss or defect judged to affect the test results, test shall not be performed or the corresponding contents shall be recorded.
(4) The mass shall be measured using a scale with graduations smaller than 0.25% of the mass. The mass shall be measured after cleaning of all the water at the surface of the specimen.

## 2.4 Testing Devices

(1) Compression Testing Machine
The testing device shall be superior to Class 1 specified in Table 1 (Classification of Testing Machine) of KS B 5533. The peak load shall range from 1/5 of weighing to the weighing, and displacement control shall be possible.
(2) Upper and bottom loading plates
The size of the upper and bottom loading plates shall be larger than the diameter of the specimen, and their thickness shall be larger than 25 mm. The loading plate shall have a spherical sheet of which center is above the surface. The loading plate shall secure rotation angle larger than 3°.
(3) Installation of safety net
Appropriate safety device shall be installed to prevent flying of potentially dangerous concrete fragments and splinters during failure.

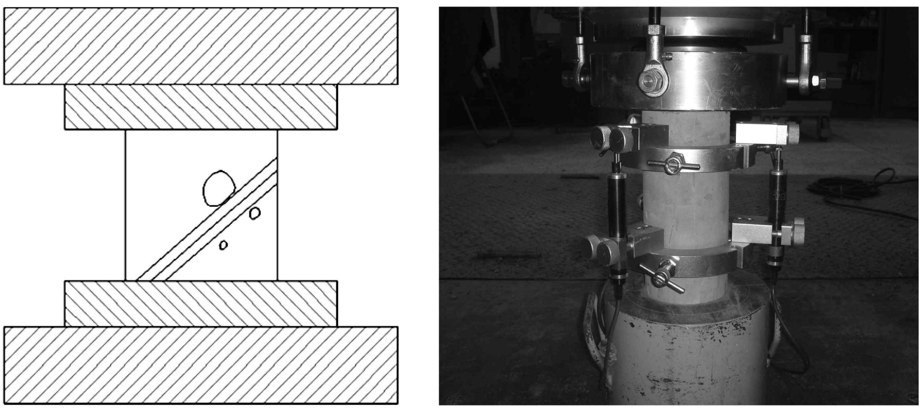


**Fig. A.2.4.1** Compressive strength test of K−UHPC

## 2.5 Testing Method

(1) The top and bottom surfaces of the specimen and the compression surfaces of the upper and bottom loading plates shall be cleaned.
(2) The specimen shall be disposed so that the central axis of the specimen coincides with the central axis of the loading plates with an error below 1% of the diameter of the specimen.
(3) During compression test, displacement can be measured by installing 3 displacement sensors (LVDT: Linear Variable Differential Transducers) longitudinally in a region within 100 mm from the center of the specimen with spacing of 120° (Figure A.2.4.1).
(4) Loading shall be applied at constant speed so as to avoid any impact on the specimen. Loading shall preferably be applied through displacement control at speed of 0.015±0.005 mm/sec.
(5) The peak load displayed by the testing machine until failure of the specimen shall be read up to 3 significant figures.
(6) The compressive strength shall be the average value obtained from more than 3 specimens.

## 2.6 Calculation Method of Results

The compressive strength shall be calculated using the following equation with 3 significant figures.

$$f_c = \frac{4P_{\max}}{\pi d^2} \quad \text{(A.2.6.1)}$$

where
$f_c$ : compressive strength (MPa)
$P_{\max}$ : peal load obtained in 2.5(5) (N)
$d$ : diameter of specimen (mm)

When the displacement is measured during the test, the compressive stress-strain curve and elastic modulus of K-UHPC can be computed. The compressive stress is computed using Equation (A.2.6.2) during the computation of the stress-strain curve.

$$\sigma = \frac{4P}{\pi d^2} \quad \text{(A.2.6.2)}$$

where
$\sigma$ : compressive stress (MPa)
$P$ : test load (N)
$d$ : diameter of specimen (mm)

In addition, the strain is computed using Equation (A.2.6.3).

$$\epsilon = \frac{\Delta}{L} \quad \text{(A.2.6.3)}$$

where
$\epsilon$ : strain
$\Delta$ : average of displacements measured by 3 displacement sensors (mm), $\Delta = (\Delta_1 + \Delta_2 + \Delta_3)/3$

$L$ : distance between displacement sensors (=100 mm)

The elastic modulus is computed by Equation (A.2.6.4).

$$E_c = \frac{\sigma_2 - \sigma_1}{\epsilon_1 - \epsilon_2} \tag{A.2.6.4}$$

where

$E_c$ : elastic modulus (MPa)

$\sigma_2$ : compressive stress corresponding to 40% of the ultimate compressive strength (MPa)

$\sigma_1$ : compressive stress corresponding to 10% of the ultimate compressive strength (MPa)

$\epsilon_2$ : strain corresponding the compressive stress $\sigma_2$

$\epsilon_1$ : strain corresponding the compressive stress $\sigma_1$

## 2.7 Report

Report shall mention the necessary items among the followings.

(1) Date of test
(2) Designation number of specimens
(3) Age
(4) Curing method and curing temperature
(5) Diameter of specimens
(6) Peak load
(7) Compressive strength
(8) Stress-strain curves (when displacement is measured)
(9) Elastic modulus (when displacement is measured)
(10) Eventual presence of defects and relevant contents
(11) Failure conditions of specimens
(12) Bulk (or apparent) density

## 3.1 Scope of Application

This standard applies for the flexural strength test method of K-UHPC specimens.

## 3.2 Quoted Standards

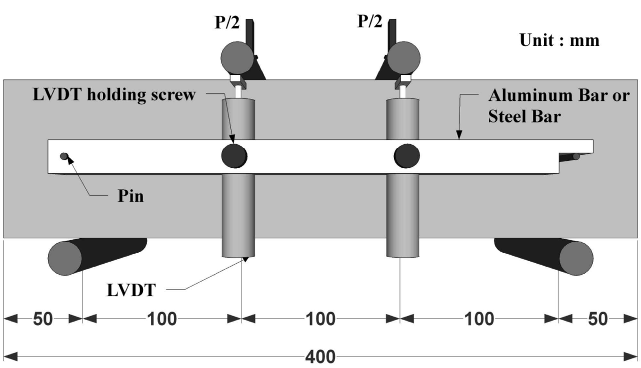
KS B 5533 Compression Testing Machines
KS F 2408 Standard Test Method for Flexural Strength of Concrete
KS F 2566 Test Method for Flexural Toughness of Steel Fiber Reinforced Concrete
ASTM C 1609 Standard Test Method for Flexural Performance of Fiber-Reinforced Concrete (Using Beam with Third-Point Loading)

## 3.3 Specimen

The fabrication of the specimen shall comply with "Appendix 1: Fabrication Method of Specimens for Strength Test of K-UHPC".

## 3.4 Testing Devices

(1) Testing Machine
The testing device shall be superior to Class 1 specified in Table 1 (Classification of Testing Machine) of KS B 5533. The peak load shall range from 1/5 of weighing to the weighing, and displacement control shall be possible.
(2) Flexural Test Device
The device for third-point loading shall not present a structure restraining even slightly the deformation of the specimen until the prescribed deflection. The adopted supports shall be up-and-down 1 set of rollers enabling rotation in the longitudinal direction.
(3) Deflection Measurement Device
The deflection measuring device used for obtaining the load-deflection curve of the specimen shall be composed by electric displacement sensors and their fixation. A displacement sensor with accuracy larger than 1/1000 mm shall be used. This is a standard to dispose two displacement sensors at the loading points. Figure A.3.4.1 illustrates one example of the testing device principle composed of 2 support rollers and 2 loading rollers.



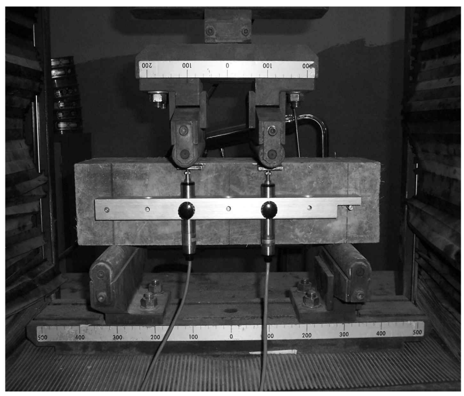



Fig. A.3.4.1 Flexural strength test of K−UHPC

## 3.5 Testing Method

(1) The specimen shall be in contact with the upper loading device at third-points of the span with reference to the width centers of the supports. At that time, there shall be no gap between the contact surface of the loading device and the specimen.
(2) The span length shall be 3 times the height of the specimen.
(3) Loading shall be applied at constant speed so as to avoid any impact on the specimen. Loading shall be applied through displacement control at speed of 0.015±0.005 mm/sec.
(4) The peak load displayed by the testing machine until failure of the specimen shall be read up to 3 significant figures.
(5) The width and height of the failed cross-section shall be measured respectively at 3 spots and 2 spots up to 0.1 mm so as to obtain the corresponding averages up to 4 significant figures.
(6) The average values obtained from more than 3 specimens shall be used as flexural strength and flexural toughness.

## 3.6 Calculation Method of Results

(1) Flexural strength
When the specimen fails between the third points of the centerline in the span direction at the tension face, the flexural strength shall be obtained by the following Equation (A.3.6.1) up to 3 significant figures. However, when the specimen fails outside the third points of the centerline in the span direction at the tension face, this results becomes invalid.

$$f_r = \frac{P_{\max} L}{bh^2} \tag{A.3.6.1}$$

where
$f_r$ : flexural strength (MPa)
$P_{\max}$ : peak load obtained in 3.5(4) (N)
$L$ : span length (mm)
$b$ : width of failed cross-section (mm)
$h$ : height of failed cross-section (mm)

(2) Flexural toughness
① The initial cracking spot of K-UHPC is defined as the LOP(Limit of Proportionality) and, the maximum flexural strength spot is defined as the MOR(Modulus of Rupture) as shown in Figure A.3.6.1. The value of the load at MOR is $P_{MOR}$ and the corresponding deflection is defined as $\delta_{MOR}$.
② The formula of the equivalent flexural strength $f_{MOR}$ at MOR is expressed in Equation (A.3.6.2). The energy $Tough_{MOR}$ up to MOR in the load-deflection curve is defined by the area of the flexural load-deflection curve. Considering the outstanding ductile behavior of K-UHPC, the flexural toughness is defined for 3 points including the two points at L/600 and L/150 and the

additional point at L/100 in order to analyze the deflection hardening behavior.

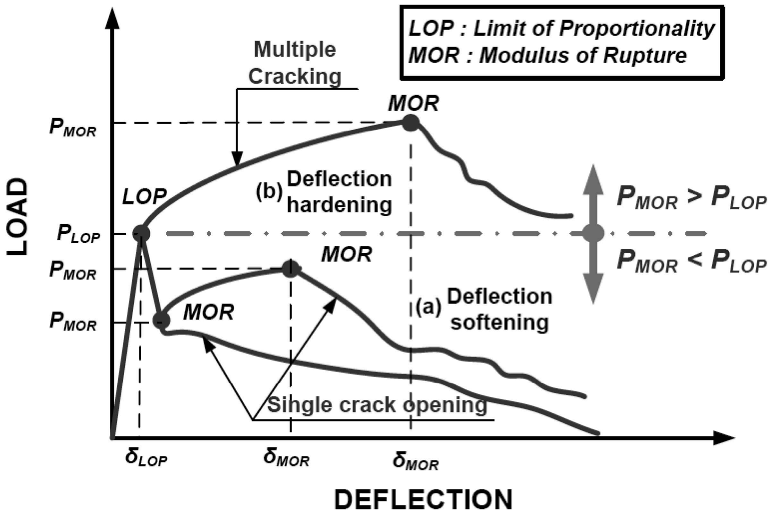



Fig. A.3.6.1 Typical load-deflection curves of K-UHPC under flexural tension stress

$$f_{MOR} = \frac{P_{MOR} L}{bh^2} \tag{A.3.6.2}$$

where

$L$ : span length (mm)

$b$ : width of specimen (mm)

$h$ : height of specimen (mm)

L/600 : clear deflection identical to L/600 of span length (0.5 mm)

L/150 : clear deflection identical to L/150 of span length (2 mm)

L/100 : clear deflection identical to L/100 of span length (3 mm)

## 3.7 Report

(1) Number of specimens
(2) Age
(3) Height, width and weight of specimens
(4) Curing method and curing temperature
(5) Measurement position of deflection
(6) Peak load
(7) Early cracking strength and deflection
(8) Peak flexural strength and deflection
(9) Equivalent flexural strength
(10) Flexural toughness

## 4.1 Scope of Application

This standard applies for the direct tensile strength test method of K-UHPC specimens.

## 4.2 Quoted Standards

KS B 5533 Compression Testing Machines

## 4.3 Specimen

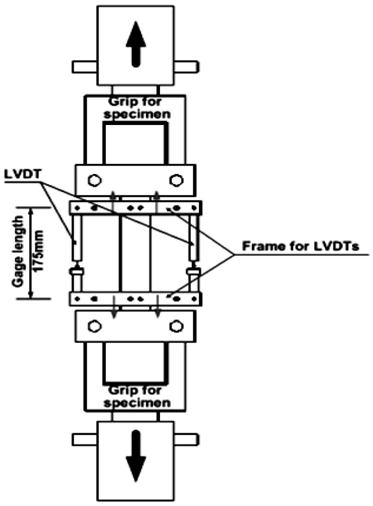
The shape, dimensions and fabrication of the specimens shall comply with "Appendix 1: Fabrication Method of Specimens for Strength Test of K-UHPC".

## 4.4 Testing Devices

(1) Testing Machine
The testing device shall be superior to Class 1 specified in Table 1 (Classification of Testing Machine) of KS B 5533. The peak load shall range from 1/5 of weighing to the weighing, and displacement control shall be possible.
(2) Tensile Test Device
The grip shall be fitted to the shape of the specimen and test load. The grip shall make the line of action of the load coincide with the axial line of the specimen. At both ends of the grip device, one end shall be fixed and one end shall be hinged (pinned) for loading.
(3) Displacement Measurement Device
The device measuring the displacement between the gauges shall be able to measure the displacement of the specimen with a precision larger than 1/1000 mm. The device shall present a structure that does not obstruct even minimally the deformation between the gauges.

## 4.5 Testing Method

(1) The loading method shall apply load on the specimen using a grip device suitable to the shape of the specimen. Figure A.4.5.1 illustrates an example of testing device.
(2) Loading shall be applied at constant speed so that the deformation speed of the specimen is 0.0050±0.0015 mm/sec.
(3) The tensile strength of K-UHPC shall be acquired when failure occurs within a constant range of 175 mm using a specimen with cross-sectional dimensions of 50×100 mm. The average of the test values shall be obtained from more than 3 specimens.



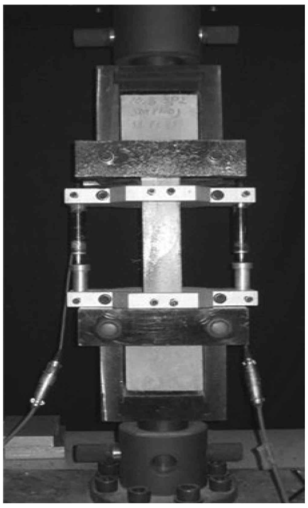
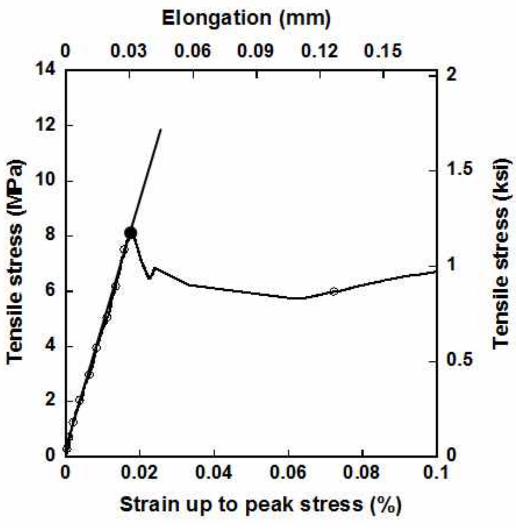



Fig. A.4.5.1 Direct tensile strength test of K−UHPC (unnotched specimen)

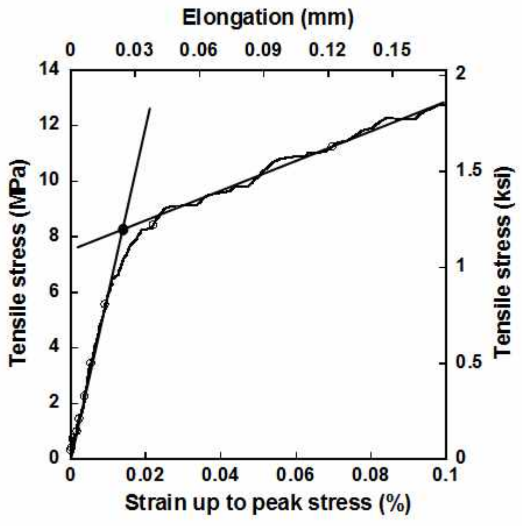
(4) Displacement sensors shall be installed at both sides of the specimen to measure the elongation. The tensile strain shall be computed using the average of the displacement sensors.
(5) The peak load displayed by the testing machine until failure of the specimen shall be read up to 3 significant figures.

## 4.6 Calculation Method of Results

(1) Early crack strength
Figure A.4.6.1 shows the method determining the early crack strength in K-UHPC. The early crack strength is selected at the point in the stress-strain curve at which the stress decreases after the initiation of crack (Figure A.4.6.1(a)). In the case where the stress does not decrease after early cracking, two lines shall be drawn in the elastic region and hardening region and, the intersection point of these two lines is defined as the early crack strength (Figure A.4.6.1(b)).




(a) Strain−softening




(b) Strain−hardening

**Fig. A.4.6.1** Method for the determination of early crack strength of K−UHPC

(2) Direct tensile strength

When failure occurs between the displacement sensors for measuring the elongation of the specimen, the tensile strength shall be calculated using the following Equation (A.4.6.1) up to 3 significant figures. However, this result is invalid when failure occurs outside the displacement sensors for measuring the elongation of the specimen.

$$f_{pc} = \frac{P_{\max}}{bh} \qquad \text{(A.4.6.1)}$$

where

$f_{pc}$ : direct tensile strength (MPa)

$P_{\max}$ : peak load indicated by the testing machine (N)

$b$ : width of failed cross-section (mm)

$h$ : height of failed cross-section (mm)

## 4.7 Report

(1) Number of specimens

(2) Age

(3) Height, width and weight of specimens

(4) Curing method and curing temperature

(5) Peak load

(6) Early cracking strength

(7) Direct tensile strength

## 5.1 Scope of Application

This standard applies for the direct tensile strength test method of K-UHPC specimens.

## 5.2 Quoted Standards

KS B 5533 Compression Testing Machines

## 5.3 Specimen

The shape, dimensions and fabrication of the specimens shall comply with “Appendix 1: Fabrication Method of Specimens for Strength Test of K-UHPC”.

## 5.4 Testing Devices

(1) Testing Machine
The testing device shall be superior to Class 1 specified in Table 1 (Classification of Testing Machine) of KS B 5533. The peak load shall range from 1/5 of weighing to the weighing, and displacement control shall be possible.
(2) Tensile Test Device
The grip shall be fitted to the shape of the specimen and test load. The grip shall make the line of action of the load coincide with the axial line of the specimen. The boundary conditions of the testing device shall hinge (pin) both ends of the direct tensile test specimen.
(3) Displacement Measurement Device
The jig of the clip gage in the notched part shall be attached to the direct tension specimen using an adhesive. The opening displacement shall be measured by installing the clip gages on the attached jig. At that time, the tension load shall be measured as the value of the load indicated by the load cell installed in the actuator.

## 5.5 Testing Method

(1) Clip gage clips shall be attached to the direct tension specimen (Figure A.5.5.1).
(2) The clip gages shall be installed after fixing the specimen axially so as to avoid eccentricity between the direct tension specimen and the load of the actuator.
(3) The displacement control speed of the actuator shall be 0.0050±0.0015 mm/sec.
(4) The load and displacement shall be measured from the start to the completion (corresponding to the reduction to 80% of the peak load) of the test.

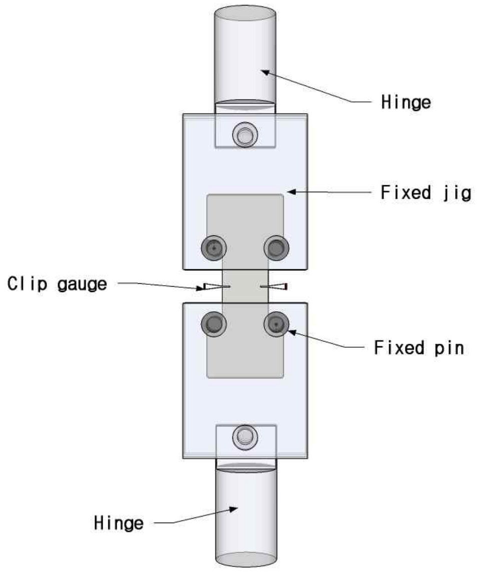



Fig. A.5.5.1 Direct tensile strength test of K-UHPC (notched specimen)

## 5.6 Calculation Method of Results

The tensile stress-opening displacement curve of the direct tension specimen is shown in Figure A.5.6.1. The tensile strength of Figure A.5.6.1 is the value obtained by dividing the tensile load of the direct tension specimen by the cross-sectional area at the notch, as expressed in Equation (A.5.6.1).

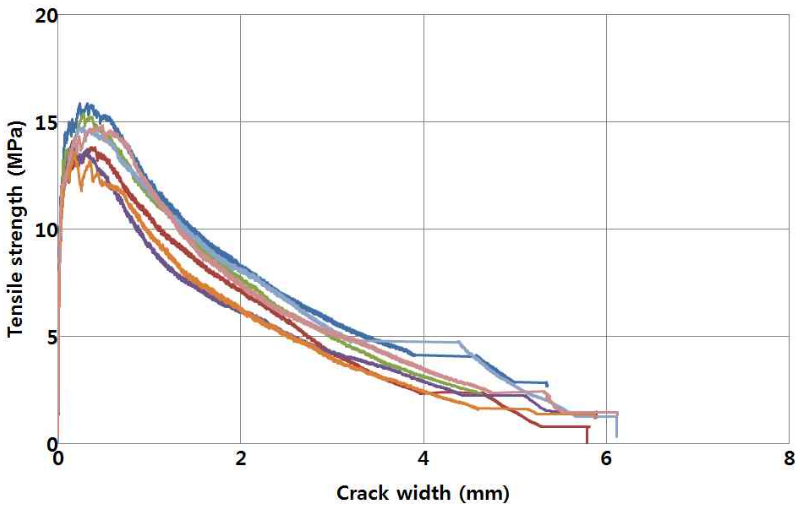



Fig. A.5.6.1 Tensile stress-opening displacement curve

$$f_{pc} = \frac{P_{\max}}{bh} \qquad \text{(A.5.6.1)}$$

where

$f_{pc}$ : direct tensile strength (MPa)

$P_{\max}$ : peak load indicated by the testing machine (N)

$b$ : width of failed cross-section (mm)

$h$ : height of failed cross-section (mm)

The computation of the early crack strength shall follow the calculation method of early crack strength (4.6(1)) of “Appendix 4: Direct Tensile Strength Test Method of K-UHPC (Unnotched Specimen)”.

## 5.7 Report

(1) Number of specimens
(2) Age
(3) Height, width and weight of specimens
(4) Width and depth of notch
(5) Curing method and curing temperature
(6) Peak load
(7) Early cracking strength
(8) Direct tensile strength
(9) Tensile stress-opening displacement

## 6.1 Single Crack State

If the external tension load increases to exceed the virtual crack stress in the steel fiber-reinforced concrete tension member, the member starts to exhibit softening behavior according to the softening of concrete and the stress increase of the reinforcement. Unlike the single crack state without steel fiber, the virtual crack stress $\sigma^i_{cf,cr}$ of the steel fiber-reinforced concrete shall be used instead of the tensile strength $f_{ct}$ of the concrete matrix.

Assuming that the cross-section remains plane even after deformation, the distribution of the whole tension force transmitted separately through the reinforcing bar and the steel fiber satisfies the compatibility conditions only by considering not only the relative displacement between the reinforcing bar and concrete but also the relative displacement between the steel fiber and concrete. Moreover, the average strains $\epsilon_{sm}$ and $\epsilon_{cm}$ of the reinforcing bar and concrete over the load transfer region $l_{es}$ can be obtained identically to the method defined for concrete reinforced only by reinforcing bar. Figure A.6.1.1 depicts intuitively the strain distribution of the reinforcing bar, steel fiber and concrete matrix passing through the single crack.

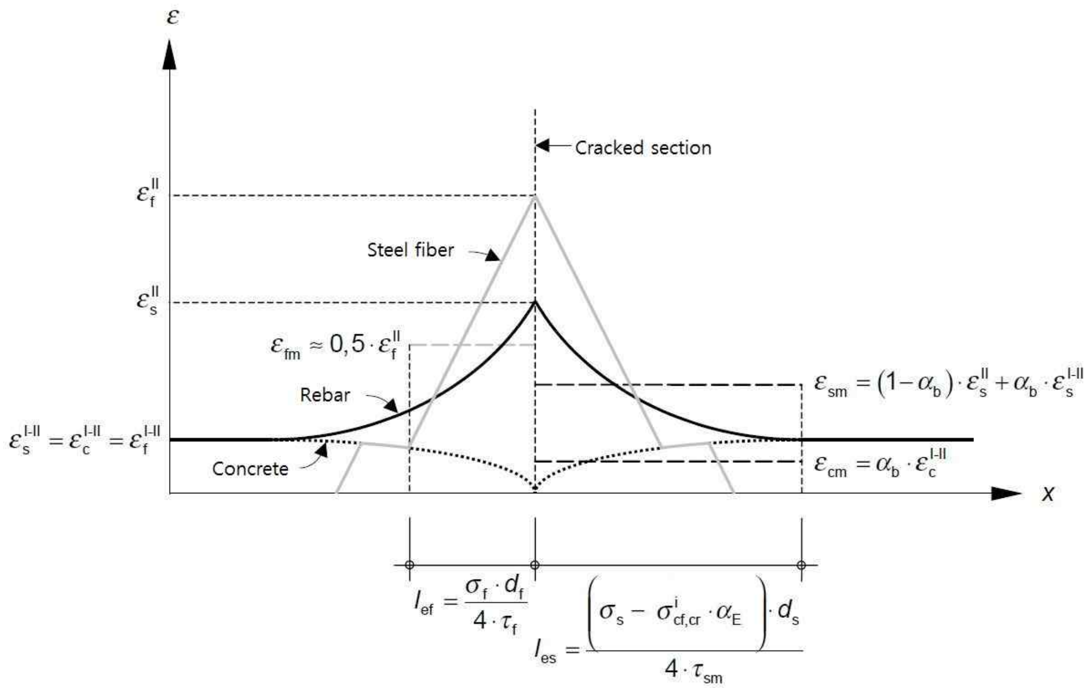



**Fig. A.6.1.1** Strain of reinforcing bar, steel fiber and concrete under single crack state

The indices "I-II" indicated in Figure A.6.1.1 express the micro-crack state, corresponding to the region extending from the no crack state to the macro-crack state in which complete cracking occurs. The corresponding strain is the strain at virtual crack stress $\sigma^i_{cf,cr}$ of the steel fiber-reinforced concrete. In addition, the value of 0.6 for $\alpha_b$ in Figure A.6.1.1 is used by assuming a parabolic shape for the strain. Since the load transfer region of the reinforcing bar extends over $2l_{es}$, the crack width in single crack state is obtained as follows.

$$
\begin{aligned}
w &= 2l_{es}(\epsilon_{sm} - \epsilon_{cm}) \\
&= \frac{(\sigma^i_{cf,cr} - \sigma_{cf})d_s}{5E_s\tau_{sm}\rho_s^2}\left[(1+\alpha_E\rho_s)\sigma^i_{cf,cr} - \sigma_{cf}\right]
\end{aligned}
\qquad \text{(A.6.1.1)}
$$

In addition, the stress $\sigma^{i}_{cf,cr}$ developed at the occurrence of the virtual crack of Equation (A.6.1.1) is expressed as follows.

$$\sigma^{i}_{cf,cr} = f_{ct}\left(1 - \frac{w^{*} f_{ct}}{2G_F}\right) + \sigma_{cf0}\left(2\sqrt{\frac{w^{*}}{w_0}} - \frac{w^{*}}{w_0}\right) \tag{A.6.1.2}$$

where

$G_F$ : fracture energy of concrete matrix without steel fiber (N/m) (when not specified separately, use 60 N/m)

$w_0$ : crack width at occurrence of the steel fiber efficiency $\sigma_{cf0}$ (mm)

$$w_0 = \frac{\tau_{fm} l_f^2}{E_f d_f}$$

$w^{*}$ : crack width at occurrence of virtual crack stress (mm)

$$w^{*} = \frac{w_0}{\left(1 + \frac{w_0 f_{ct}^2 g}{2\sigma_{cf0} G_F}\right)^2}$$

$\sigma_{cf0}$ : efficiency of steel fiber (MPa) (maximum tensile stress that can be transferred by the steel fiber)

$$\sigma_{cf0} = \eta g \rho_f \frac{\tau_{fm} l_f}{d_f}$$

$f_{ct}$ : tensile strength of concrete matrix without steel fiber (MPa) ($f_{ct} \cong f_{ctm}$, when not specified separately, use 8.5 MPa)

$d_s$ : diameter of reinforcement (mm)

$\rho_s$ : reinforcement ratio

$\tau_{fm}$ : average bond stress between individual steel fiber and concrete (MPa) (when not specified separately, use 6.8 MPa)

$l_f$ : length of steel fiber (mm)

$d_f$ : diameter of steel fiber (mm)

$\rho_f$ : steel fiber ratio

$E_f$ : elastic modulus of steel fiber (MPa)

$E_s$ : elastic modulus of reinforcing bar (MPa)

$g$ : efficiency factor of steel fiber (use 1.13 if not specified)

$\eta$ : orientation factor of steel fiber ($b$ is the maximum dimension of the placed member cross-section (mm), $\eta_{1D}$ = 1.0 and $\eta_{2D}$ = 0.637)

$$\eta = \frac{\eta_{1D} l_f + \eta_{2D}(b - l_f)}{b}$$

The tensile stress $\sigma_{cf}$ of steel fiber-reinforced concrete of Equation (A.6.1.1) is obtained using Equation (A.6.1.3). This means that the tensile stress is function of the crack width and, therefore, that the calculation of the crack width using Equation (A.6.1.1) can be done only through iterative calculation.

(a) Fiber action phase:

$$\sigma_{cf} = \sigma_{cf0}\left(2\sqrt{\frac{w}{w_0}} - \frac{w}{w_0}\right) \tag{A.6.1.3-a}$$

(b) Fiber pull-out phase:

$$\sigma_{cf} = \sigma_{cf0}\left(1 - 2\frac{w}{l_f}\right)^2 \tag{A.6.1.3-b}$$

The average bond stress $\tau_{sm}$ of the reinforcing bar in Equation (A.6.1.1) is obtained as follows.

$$\tau_{sm} = \frac{\tau_{b,\max}}{(1+\lambda\alpha)}\left(\frac{w}{2s_1}\right)^{\alpha} \tag{A.6.1.4}$$

where

$$\lambda = \begin{cases} \dfrac{2}{(1-\alpha)} & F \le F_{cr} \\ \left(\dfrac{1+\alpha}{1-\alpha}\right)\left(\dfrac{F_{cr}}{F}\right)^{1.5} + 1.0 & F > F_{cr} \end{cases}$$

$F$ : external tensile load (N)

$F_{cr}$ : tensile load at occurrence of single crack (N)

$\tau_{b,\max}$ : maximum bond stress between reinforcing bar and K-UHPC (MPa) (46.5 MPa for reinforcing bar < D19, 58.9 MPa for reinforcing bar > D19)

$s_1$ : slip of reinforcing bar occurring at $\tau_{b,\max}$ (mm), use 0.4 mm

$\alpha$ : nonlinear index (0.15 for reinforcing bar < D19, 0.14 for reinforcing bar > D19)

## 6.2 Steady Cracking State

Similarly to the steady cracking state in concrete reinforced by reinforcing bar and without steel fiber, the steady cracking state in concrete reinforced by steel fiber and reinforcing bar is also the state expressing the difference in strain between concrete and the reinforcing bar all over the member as shown in Figure A.6.2.1.

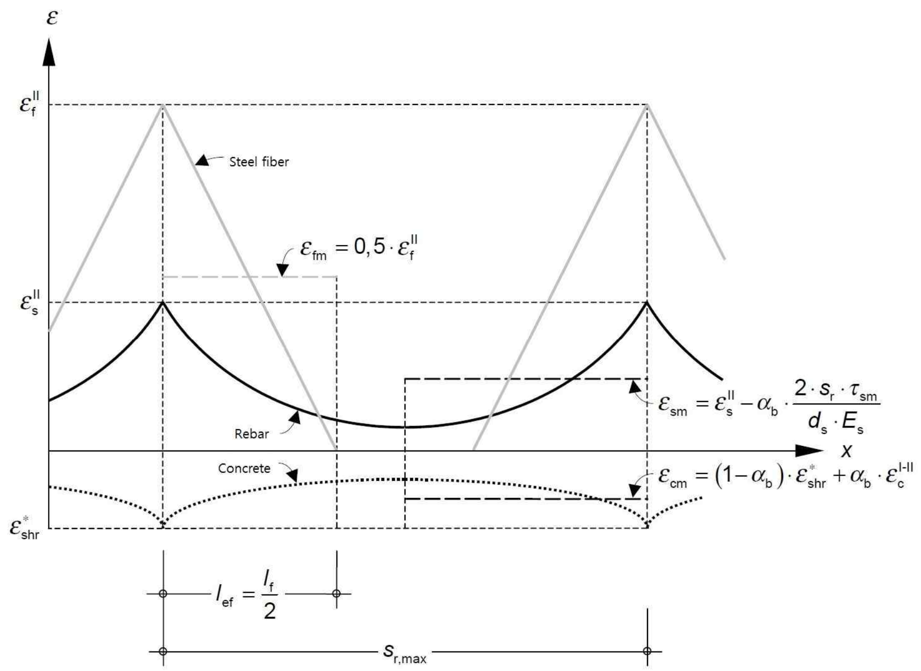



Fig. A.6.2.1 Strain of reinforcement, steel fiber and concrete in steady state cracking

The crack width in steady state cracking passing through the maximum crack spacing $S_{r,\max}$ shall be obtained as follows.

$$\begin{aligned} w_{\max} &= S_{r,\max}(\epsilon_{sm} - \epsilon_{cm}) \\ &= \frac{(\sigma^i_{cf,cr} - \sigma_{cf})d_s}{2E_s\tau_{sm}\rho_s}\left[\sigma_s - 0.6\frac{\sigma^i_{cf,cr}}{\rho_s}(1+\alpha_E\rho_s) + 0.6\frac{\sigma_{cf}}{\rho_s} - 0.4E_s\epsilon^*_{s,shr}\right] \end{aligned} \qquad (부\ 6.2.1)$$

where

$S_{r,\max}$ : maximum crack spacing (mm), $S_{r,\max} = \dfrac{(\sigma^i_{cf,cr} - \sigma_{cf})d_s}{2\tau_{sm}\rho_s}$ .

$\epsilon^*_{s,shr}$ : concrete shrinkage strain at the surface of the cracked part after occurrence of cracking, which considers the relaxation caused by the shrinkage and creep of concrete. Approximately equal to the free shrinkage coefficient $\epsilon_{cs}$ of concrete (when not specified separately, use –0.001).

❖ Authors

| Author | Name | Affiliation |
|---|---|---|
| Executive authors | Kim, Jee-Sang | Seokyeong University |
| | Kim, Byung-Suk | Korea Institute of Construction Technology |
| Authors | Jeon, Se-Jin | Ajou University |
| | Koh, Kyung-Taek | Korea Institute of Construction Technology |
| | Kwahk, Imjong | Korea Institute of Construction Technology |
| | Kim, Sung-Wook | Korea Institute of Construction Technology |
| | Kim, Young-Jin | Korea Institute of Construction Technology |
| | Park, Sung-Yong | Korea Institute of Construction Technology |
| | Joh, Changbin | Korea Institute of Construction Technology |