CHAPTER 1: GENERAL

1.1 SCOPE

(1) This Design Recommendation gives general requirements for design of concrete structures using continuous fiber reinforcing materials. Subjects not covered in this Recommendation shall comply with the JSCE "Standard Specification for Design and Construction of Concrete Structures (Design)" (hereinafter referred as the "JSCE Standard Specification (Design)").

(2) Continuous fiber reinforcing materials shall in principle conform to JSCE-E 131 "Quality Specifications for Continuous Fiber Reinforcing Materials".

[COMMENT]:

(1) "Concrete structures using continuous fiber reinforcing materials" include structures where continuous fiber reinforcing materials is used together with steel reinforcement or prestressing steel. Chapter and section numbers given in this Design Recommendation refer to the JSCE Standard Specification (Design), 1996 edition.

1.2 DEFINITIONS

The following terms are defined for general use in this Design Recommendation:

Reinforcing materials: Materials used to reinforce concrete. These include steel and continuous fiber reinforcing materials.

Continuous fiber: General term for continuous fibers used to reinforce concrete. These include carbon fibers, Aramid fibers and glass fibers.

Fiber binding materials: Adhesive used to consolidate continuous fibers. These are mostly plastics such as epoxy resin or vinylester resin.

Continuous fiber reinforcing materials (CFRM): General term for unidirectional reinforcement formed from continuous fibers impregnated with a fiber binding material, then hardened and molded, in the form of bundled or woven continuous fibers, used for reinforcing the concrete.

Capacity of CFRM: Maximum load that a continuous fiber reinforcing material can sustain.

Strength of CFRM: Value obtained by dividing the capacity of continuous fiber reinforcing material by the nominal cross-sectional area.

Characteristic value of capacity of CFRM: Value for the capacity of continuous fiber reinforcing material below which the percentage of test results obtained using a given test method is guaranteed not to exceed a given figure, allowing for variations in test results.

Specified value of capacity of CFRM: (as distinct from the characteristic value of capacity of CFRM:) Capacity value for continuous fiber reinforcing material determined in accordance with structural specifications other than this Recommendation, or other regulations.

Guaranteed capacity of CFRM: Guaranteed capacity in accordance with JSCE-E 131 "Quality Specifications for Continuous Fiber Reinforcing Materials"

Design capacity of CFRM: Value obtained by dividing the characteristic value of capacity of continuous fiber reinforcing material by the material coefficient.

Characteristic value of ultimate strain of CFRM: Strain corresponding to the characteristic value of tensile capacity of continuous fiber reinforcing material.

Design ultimate strain of CFRM: Value obtained by dividing the characteristic value of ultimate strain of continuous fiber reinforcing material by the material coefficient.

Tensile rigidity of CFRM: Slope of the tensile force-strain curve for continuous fiber reinforcing material, when this curve is assumed to be linear.

Young's modulus of CFRM: Value obtained by dividing the tensile rigidity of continuous fiber reinforcing material by the nominal cross-sectional area.

Nominal cross-sectional area of CFRM: Value obtained by dividing the volume of continuous fiber reinforcing material by the length.

Bent portion of CFRM: Portion of continuous fiber reinforcing material set in a curved shape by hardening with a fiber binding material while the continuous fibers are bent.

Curved placement of CFRM: Placement of straight continuous fiber reinforcing material in a curved layout.

Creep failure: Failure due to progressive loss of tensile capacity over time, when continuous fiber reinforcing material is subjected to a continuous static tensile load.

Creep failure capacity: Capacity at the time of creep failure.

Flexural compressive failure: Form of failure in members subjected to flexure, whereby the compressed section of concrete fails before the main continuous fiber reinforcing material breaks.

Fiber rupture flexural failure: Form of failure in members subjected to flexure, whereby the main continuous fiber reinforcing material breaks before the failure of the compressed section of concrete.

Fiber rupture shear failure: Form of shear failure in members subject to shear forces, due to breaking of continuous fiber reinforcing material used as shear reinforcement.

[COMMENTS]:

Definitions of shear reinforcement, lateral ties, hoop reinforcement, spiral reinforcement, and tendons follow the JSCE Standard Specification (Design), where "steel reinforcement or prestressing steel" shall be read simply as "reinforcing materials".

Since the nominal cross-sectional area of CFRM is obtained by dividing the volume of the CFRM by the length, and volume generally includes sectional area which does not contribute to the strength of the reinforcement, the strength and Young's modulus of CFRM obtained by division by the nominal cross-sectional area are not identical with the value for the continuous fiber itself.

1.3 NOTATION

Notation used in this Design Recommendation with reference to structural design is as follows:

A_f : Cross-sectional area of CFRM placed in tensile zone
A_{fc} : Cross-sectional area of CFRM necessary based on calculation
c_f : Center-to-center distance of CFRM
E_0 : Standard Young's modulus (200 kN/mm^2 = Young's modulus of steel)
E_f : Young's modulus of CFRM used in verification of service limit state
E_{fp} : Young's modulus of CFRM used as tendons
E_{fu} : Young's modulus of CFRM used in verification of ultimate limit state
E_w : Young's modulus of shear reinforcement or transverse torsional reinforcement
F_{fu} : Tensile capacity of CFRM
f_{fb} : Strength of bent portion of CFRM
f_{fc} : Creep failure strength of CFRM
f_{fpu} : Tensile strength of CFRM used as tendons
f_{fu} : Tensile strength of CFRM
f_w : Strength of shear reinforcement or transverse torsion reinforcement
γ_{mf} : Material coefficient of CFRM
ε_{fspd} : Design value of strain at ultimate limit state for spiral reinforcement
ε_{fu} : Ultimate strain of CFRM
ε_{fwd} : Design value of strain at ultimate limit state for shear reinforcement
σ_{fe} : Increase in reinforcement stress due to design load, used in verification of crack width
σ_{fp} : Increase in reinforcement stress due to permanent load
σ_{fpe} : Increase in tendon stress due to design load, used in verification of crack width
σ_{fpp} : Increase in tendon stress due to permanent load

[COMMENT]:

Subscript *f* refers to CFRM.

CHAPTER 2: DESIGN BASICS

2.1 GENERAL

It shall be in accordance with JSCE Standard Specification (Design), section 2.1.

2.2 DESIGN SERVICE LIFE

It shall be in accordance with JSCE Standard Specification (Design), section 2.2.

2.3 DESIGN PREREQUISITE

It is assumed for the purposes of design based on this Recommendation that construction on site will be carried out appropriately at all times.

[COMMENT]:
The basic stance relating to structural design is given here. It is assumed that construction is carried out following the intentions of the designer. Appropriate construction refers to construction carried out according to the Construction Recommendation.

2.4 DESIGN PRINCIPLES

It shall be in accordance with JSCE Standard Specification (Design), section 2.4.

2.5 CALCULATION OF SECTIONAL FORCE AND CAPACITY

It shall be in accordance with JSCE Standard Specification (Design), section 2.5.

2.6 SAFETY FACTORS

It shall be in accordance with JSCE Standard Specification (Design), section 2.6. Safety factors relating to CFRM shall be determined according to each limit state.

[COMMENT]:
Standard values for safety factors are shown in **Table C 2.6.1**, below.

Table C 2.6.1: Standard safety factors

	Material factor γ_m			Member factor	Structural analysis factor	Load factor	Structural factor
	Concrete γ_c	CFRM γ_{mf}	Steel γ_s	γ_b	γ_a	γ_f	γ_i
Ultimate limit state	1.3* or 1.5	1.15** to 1.3	1.0 or 1.05	1.15 to 1.3	1.0	1.0 to 1.2	1.0 to 1.2
Serviceability limit state	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Fatigue limit state	1.3* or 1.5	1.15** to 1.3	1.05	1.0 to 1.1	1.0	1.0	1.0 to 1.1

* 1.3 where characteristic value of concrete compressive strength f'_{ck} is less then 50 N/mm^2

** 1.15 for CFRM with carbon or Aramid fibers

2.7 CORRECTION FACTOR

It shall be in accordance with JSCE Standard Specification (Design), section 2.7.

2.8 DESIGN CALCULATIONS

It shall be in accordance with JSCE Standard Specification (Design), section 2.8.

2.9 DRAWINGS

Design drawings shall give structural and reinforcement details, showing clearly the following:
(1) Design conditions
(2) Details of bent portion of CFRM
(3) Cover of reinforcing material in all parts of the structure
(4) Locations of construction joints assumed in design
(5) Detail drawings of zones with intertwining reinforcing materials, sheaths, anchor bolts etc.
(6) Nominal diameter of sheaths, if used
(7) Locations and dimensions of major chamfers

[COMMENTS]:

Design drawings should be considered the only means of transmitting the intentions of the designer to the constructor. Clear information must therefore be given regarding the conditions on which the design is based. These include the standard design strength of concrete, slump, maximum size of coarse aggregate, standards for reinforcing materials and minimum compressive strength of concrete at which prestressing may be carried out in post-tensioning prestressed concrete.

The capacity of bent portion of CFRM is generally lower than that of straight lengths, but the degree of loss depends heavily on the geometry and dimensions of the bent portion. Therefore, details of the bent portion must be given clearly. Concrete cover and concrete quality are also important factors in relation to the durability of concrete structures, and the realization of a durable concrete structure depends on these factors being examined thoroughly at the design stage. In order to transmit all of these details to the constructor, concrete cover in all parts should be clearly indicated in the design drawings.

Detail drawings of zones with intertwining reinforcing materials, sheaths, anchor bolts etc. should be prepared, and the properties of concrete at these zones be verified.

CHAPTER 3: DESIGN VALUES FOR MATERIALS

3.1 GENERAL

(1) The quality of concrete and reinforcing materials are expressed, in addition to compressive strength and tensile strength, in terms of material characteristics such as strength characteristics, Young’s modulus, deformation characteristics, thermal characteristics, durability, water tightness etc., according to the design requirements. In the case of strength and deformation characteristics, loading velocity may have to be taken into consideration.

(2) The characteristic values given for material strength and ultimate strain of CFRM are minimum values the majority of test results are guaranteed to exceed, allowing for variations in test values.

(3) Values for the design strength of materials and the design ultimate strain of CFRM shall be obtained by dividing the relevant characteristic values by the material coefficients.

[COMMENT]:
(2) It is recognized that the tensile strengths obtained from tensile tests using the same CFRM show greater variation than does steel. The amount of variation in tensile strength differs depending on the type, geometry etc. of the continuous fibers and the fiber binding material, and variation is found even for the same CFRM depending on the length of the test piece and the anchoring method used during testing. The characteristic values for the material strength of CFRM are therefore minimum values the majority of test results are guaranteed to exceed.

3.2 CONCRETE

It shall be in accordance with JSCE Standard Specification (Design), 3.2.

3.3 STEEL

It shall be in accordance with JSCE Standard Specification (Design), 3.3.

3.4 CFRM

3.4.1 Capacity

(1) Characteristic values for tensile capacity of CFRM shall be determined on the basis of tensile tests. Tensile tests shall be conducted in accordance with "Test Method for Tensile Properties of Continuous Fiber Reinforced Materials (JSCE-E 531-1995)".

(2) For materials conforming to "Quality Specifications for Continuous Fiber Reinforced Materials (JSCE-E 131)", the tensile capacity may be taken to be identical to the guaranteed capacity.

(3) Where CFRM is to be shaped by bent portion or curved placement, or where CFRM are to be subjected to diagonal tensile forces, the capacity shall be determined based on the results of suitable tests.

(4) The design strength of bent portion of CFRM shall normally be calculated as follows:

$$f_{fbd} = f_{fbk} / \gamma_{mfb} \tag{3.4.1}$$

where $$f_{fbk} = \left(0.05\frac{r}{h} + 0.3\right) f_{fuk} \tag{3.4.2}$$

If the right side of the above equation resolves to a value greater than f_{fuk}, f_{fbk} shall be taken as f_{fuk}.

f_{fbk} : characteristic value of strength of bent portion
f_{fuk} : characteristic value of unconfined tensile strength
r : internal radius of bend
h : cross-sectional height of CFRM
γ_{mfb} : can generally be taken as 1.3

(5) The design strength of CFRM to be used in a curved placement may be obtained by subtracting the elastic bending stress of the curved portion from the design strength of the straight portion.

(6) The compressive capacity and shear capacity of CFRM may be ignored for design purposes.

(7) The material coefficient γ_{mf} of CFRM shall be determined allowing for the quantity and deviation of test data, possible damage to CFRM during transportation and construction, differences in material characteristics between test pieces and actual structures, the effects of material characteristics on the limit state, service temperatures, environmental conditions etc. γ_{mf} may generally be set between 1.15 and 1.3.

[COMMENTS]:

(1) CFRM are compound materials formed from continuous fibers and fiber binding materials. When forces act on CFRM, therefore, at the microscopic level the local stresses acting on individual fibers and the binding materials will vary. When considering CFRM as reinforcing material in concrete, however, it is simpler to treat the CFRM as a monolithic material. The strength of CFRM is thus taken to be the capacity of the entire section (at maximum load). If the nominal-cross sectional area of the CFRM is known, strength (maximum load / nominal cross-sectional area) may be used instead of capacity.

(3) If CFRM are to be used in bent portion or in curved placement, or if the CFRM are subjected to diagonal tensile forces such that diagonal cracks occur, the tensile capacity falls below the unconfined tensile capacity of the straight CFRM. In bent portion or curved placement, the rate of reduction has been confirmed experimentally to be dependent on the ratio of the radius of curvature of the bent portion or curved placement and the diameter of the CFRM, on the angle of the working tensile force if diagonal tensile forces are present, etc. In such cases, the capacity shall be determined on the basis of the results of suitable tests. When CFRM are to be used in curved placement, the capacity shall

normally be determined according to "Test Method for Flexural Tensile Properties of Continuous Fiber Reinforced Materials (JSCE-E 532-1995)".

(4) The strength of bent portion varies greatly even for the same type of fiber, depending on the bending technique, type of resin used etc., therefore the strength of the bent portion will generally be determined on the basis of suitable tests. From comparisons with existing test data, the strength of bent portion has been found to be derivable as a function of the internal radius of the bent section, from Eq. (3.4.1). The regression equation in **Fig. C 3.4.1** gives the averages of all test data. The design equation Eq. (3.4.2), based on this regression formula, gives an adequate margin of safety.

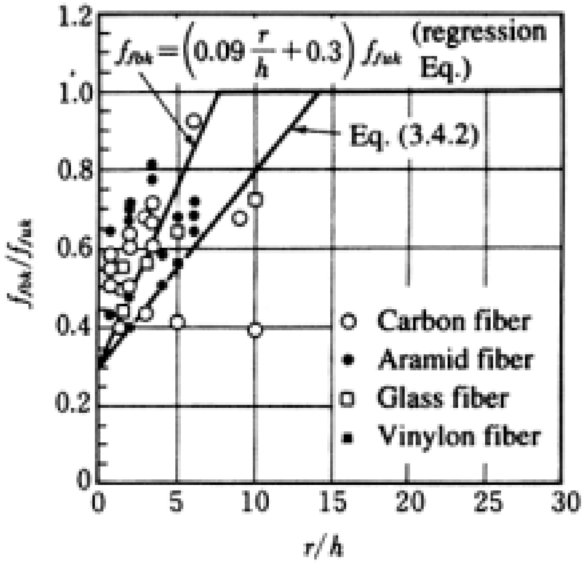


Fig. C 3.4.1: Strength of bent portion

(5) CFRM generally have a lower elastic modulus than steel reinforcement, so they can be bent and arranged within the elastic region. At small bending radii, though, the strength of the bent portion is reduced by the effects of elastic bending stress and bearing stress. The policy adopted here has been to approximate these effects by subtracting the elastic bending stress from the strength of the straight portion.

(6) CFRM consist of extremely fine collection of fibers, and therefore have extremely low compressive and shear capacity when used as reinforcing material. In normal designs, therefore, compressive and shear capacity of CFRM will be ignored.

3.4.2 Fatigue capacity

(1) The characteristic values for fatigue capacity of CFRM shall be determined based on fatigue capacity derived from fatigue tests conducted allowing for the type, size, anchoring, intensity and frequency of working stress, environmental conditions etc.

(2) The material coefficient γ_{mf} relating to the design fatigue capacity of CFRM is determined allowing for the quantity and deviation of fatigue test data, service temperatures, environmental conditions etc. If the CFRM are liable to suffer damage during transportation and construction, the effects of such damage shall be allowed for in γ_{mf}. γ_{mf} may generally be set between 1.15 and 1.3.

[COMMENTS]:

(1) The quantity of research findings relating to the fatigue in CFRM is still inadequate, and further experimental investigations are required.

When CFRM is used as tendons in prestressed concrete, if cracking is not allowed, the variable stresses will be small and the effects of fatigue will be negligible, but if cracking is allowed, fatigue must be verified in the same way as if prestress was not present. The fatigue capacity of CFRM requires the fatigue characteristics not only of the CFRM, but also of the anchorages to be clarified. As loss of capacity due to secondary stresses in particular, is significant in CFRM, the fatigue characteristics including those of the anchorages are important.
The static capacity of bent portion is known to be considerably lower than that of straight portions for certain types of CFRM. The fatigue capacity of bent portion is still lower than the static capacity of bent portion.

Where slipping of CFRM occurs at intersections with cracks etc., fatigue strength is known to be reduced even in conventional steel reinforcement, but the fatigue capacity in CFRM is reduced still further because the static capacity is also reduced. This reduction of fatigue capacity occurs at the intersections with shear cracks of both shear and tensile reinforcement.

3.4.3 Tensile force-strain relationship

(1) The tensile force-strain curve of CFRM used in verification of ultimate limit state may be assumed to follow the model shown in **Fig. 3.4.1**, in which a straight line connects tensile capacity obtained from tests and the corresponding ultimate strain points with the origin.

(2) The tensile force-strain curve used in verification of the serviceability limit state of CFRM may be assumed to follow the model shown in **Fig. 3.4.2**, in which a straight line connects the tensile rigidity calculated in accordance with "Test Method for Tensile Properties of Continuous Fiber Reinforcing Materials (JSCE-E 531-1995)".

(3) The tensile force-strain curve used in verification of the fatigue limit state of CFRM shall be the same as that used in verification of the serviceability limit state.

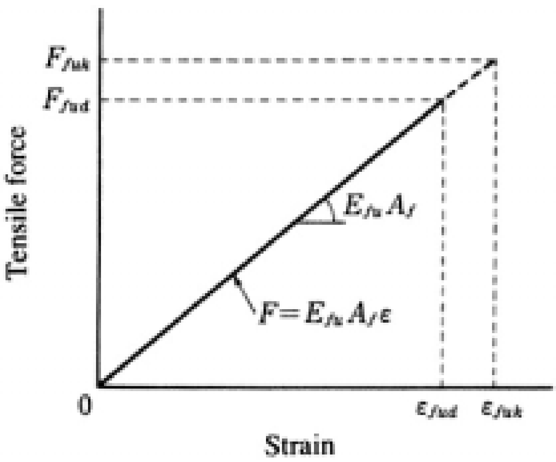


Fig. 3.4.1 Tensile force-strain curve used for the design of ultimate limit state

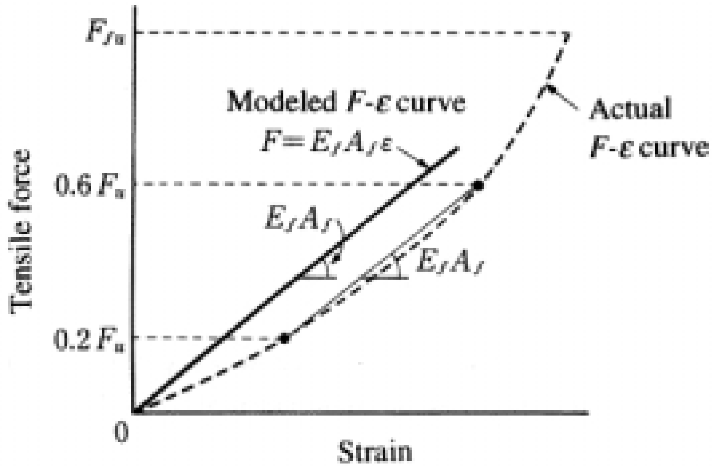


Fig. 3.4.2 Tensile force-strain curve used for the design of serviceability limit state

[COMMENTS]:

(1) The tensile force-strain curves for CFRM vary slightly depending on the type of fiber, but in general the tangential rigidity varies with the load level as shown in **Fig. 3.4.2**, therefore models have been set up for each limit state. For the tensile force-strain curve used in verification of ultimate limit state, test for tensile strength according to JSCE-E 531 is carried out and the bearing characteristics of capacity are calculated according to JSCE-E 131. The design capacity is obtained by dividing this by the material coefficient, and the design ultimate strain is obtained by dividing this by the nominal cross sectional area and Young's modulus.

(2) The tensile force-strain curve used in verification of the serviceability limit state is the tensile force-strain curve obtained according to JSCE-E 531, assumed to be a straight line through the origin having the same gradient as the line connecting the points corresponding to tensile capacity of 20% and 60%.

3.4.4 Coefficient of thermal expansion

The coefficient of thermal expansion of CFRM shall generally be as given in **Table 3.4.1**.

Table 3.4.1 Thermal expansion coefficient of CFRM

Type of CFRM	Thermal expansion coefficient ($\times 10^{-6}$/°C)
Aramid fiber	-6
Carbon fiber	0
Glass fiber	10

[COMMENT]:

The coefficients of thermal expansion of CFRM in the axial direction vary depending on the type of fiber, within the ranges shown in **Table C 3.4.1**. The values given in **Table C 3.4.1** for glass fiber are the same as those for concrete. Conservative values are given for other types of fiber, where the coefficients of thermal expansion are different from those of concrete.

Table C 3.4.1 Thermal expansion coefficient of CFRM

Type of CFRM	Thermal expansion coefficient ($\times 10^{-6}/\ ^{o}C$)
Aramid fiber	-2 ~ -6
Carbon fiber	0.6 ~ 1
Glass fiber	9 ~ 10

3.4.5 Relaxation rate

(1) Relaxation rate for CFRM shall generally be as calculated according to "Test Method for Long-Term Relaxation of Continuous Fiber Reinforcing Materials (JSCE-E 534-1995)".

(2) The apparent relaxation rate to be used in calculating prestress loss shall be based on the relaxation rate of the CFRM, allowing for the effects of drying shrinkage and creep of the concrete.

[COMMENTS]:

(1) As little data is available relating to relaxation rate of CFRM, and long-term data (more than 1000 hours) is especially lacking, it has been decided to use the values obtained according to JSCE-E 534. The relaxation rate corresponding to a service life of 100 years is taken to be the value for 1 million hours, extrapolated from the relaxation values for times in excess of 1000 hours. Where the service life of the structure is determined in advance, the relaxation value corresponding to the predetermined service life may be applied.

(2) Little experimental data is currently available on which to base an equation for the calculation of apparent relaxation rate. This may therefore be estimated on the basis of test data, or if necessary the net relaxation rate may be used.

3.4.6 Creep failure capacity

The creep failure capacity of CFRM shall be calculated according to "Test Method for Creep Failure of Continuous Fiber Reinforcing Materials (JSCE-E 533-1995)".

[COMMENT]:

CFRM subjected to sustained stresses for long periods may undergo rupture (creep failure) at less than the static bearing capacity. This creep failure capacity varies depending on the fiber type. Tensioning must therefore be carried out allowing for the creep failure capacity when CFRM is used as tendons. For design purposes, the creep failure capacity is that corresponding to a design service life of 100 years and the creep failure capacity based on the 1 million hour creep failure - limit load ratio given in JSCE-E 533 shall be applied. Where the service life of the structure is determined in advance, the creep failure capacity corresponding to the predetermined service life may be estimated from the 1 million hour creep failure - limit load ratio.

CHAPTER 4: LOADS

4.1 GENERAL

It shall be in accordance with JSCE Standard Specification (Design), 4.1.

4.2 CHARACTERISTICS VALUES OF LOADS

It shall be in accordance with JSCE Standard Specification (Design), 4.2.

4.3 LOAD FACTORS

It shall be in accordance with JSCE Standard Specifications (Design), 4.3.

4.4 LOAD TYPES

(1) Loads other than seismic loads shall be in accordance with JSCE Standard Specification (Design), 4.4.

(2) Seismic loads shall be in accordance with JSCE Standard Specifications (Seismic Design). The effects of plastic deformation of structures shall normally not be considered.

[COMMENT]:

When steel is used as reinforcing material, allowance for the effects of plastic deformation of structures due to yielding of steel members is permitted, but as yielding does not take place when CFRM is used, structures cannot be expected to undergo plastic deformation unless special measures are taken. For this reason, plastic deformation of structures shall normally not be considered. Where steel reinforcement is used in conjunction with CFRM, seismic behavior must be verified on the basis of a suitable evaluation of the plastic deformation capacity of the structure, either according to test results or to non-linear analysis based on a reliable theory.

CHAPTER 5: STRUCTURAL ANALYSIS

5.1 GENERAL

It shall be in accordance with JSCE Standard Specification (Design), 5.1.

5.2 CALCULATION OF SECTIONAL FORCES IN ULTIMATE LIMIT STATE

It shall be in accordance with JSCE Standard Specification (Design), 5.2. Redistribution of bending moment due to plastic deformation of structures shall not be considered in general.

[COMMENT]:

Allowance for redistribution of bending moment due to plastic deformation of structures is normally permitted in statically indeterminate structures incorporating continuous beams, rigid frames, continuous slabs etc. However, as yielding does not take place when CFRM is used, unless special constraining reinforcement is placed in the concrete, the structure cannot be expected to yield. For this reason, redistribution of bending moments due to plastic deformation of structures shall not be considered in general. If the rate of rigidity loss due to the appearance of cracking varies greatly between different members, the effects of redistribution of bending moments due to cracking sometimes cannot be ignored. In such cases, redistribution of bending moments due to cracking must be allowed for in calculation of section forces.

5.3 CALCULATION OF SECTIONAL FORCES AND DEFORMATION IN SERVICEABILITY LIMIT STATE

It shall be in accordance with JSCE Standard Specification (Design), 5.3.

5.4 CALCULATION OF SECTIONAL FORCES IN FATIGUE LIMIT STATE

It shall be in accordance with JSCE Standard Specification (Design), 5.4.

CHAPTER 6: ULTIMATE LIMIT STATE

6.1 GENERAL

It shall be in accordance with JSCE Standard Specification (Design), 6.1. The collapse mechanism in statically indeterminate structures shall not be considered.

[COMMENT]:
As yielding does not take place in CFRM, the collapse mechanism due to the formation of plastic hinges shall generally not be considered. The effects of steel reinforcement on member capacity when CFRM is used in conjunction with steel reinforcement may be calculated according to JSCE Standard Specification (Design), 6.2 to 6.4.

6.2 SAFETY VERIFICATION OF BENDING MOMENT AND AXIAL FORCE

6.2.1 Design capacity of member cross-section

(1) In members subjected to axial compressive force, the upper limit of axial compressive capacity N'_{oud} shall be calculated according to Eq. (6.2.1) when ties are used, and according to Eq. (6.2.1) or Eq. (6.2.2) whichever that gives the larger result when spiral reinforcement is used.

$$N'_{oud} = 0.85 f'_{cd} A_c / \gamma_b \qquad (6.2.1)$$

$$N'_{oud} = (0.85 f'_{cd} A_e + 2.5 E_{sp} \varepsilon_{fspd} A_{spe}) / \gamma_b \qquad (6.2.2)$$

where

A_c : cross-sectional area of concrete

A_e : cross-sectional area of concrete enclosed by spiral reinforcement

A_{spe} : equivalent cross-sectional area of spiral reinforcement ($=\pi d_{sp} A_{sp}/s$)

d_{sp} : diameter of concrete section enclosed by spiral reinforcement

A_{sp} : cross-sectional area of spiral reinforcement

s : pitch of spiral reinforcement

f'_{cd} : design compressive strength of concrete

E_{sp} : Young’s modulus of spiral reinforcement (E_{fu})

ε_{fspd} : design value for strain of spiral reinforcement in ultimate limit state, may generally be taken as 2000×10^{-6}. If the design strength f_{fbd} is less than $E_{sp}\varepsilon_{fspd}$ when the spiral reinforcement is regarded as a bent portion, $E_{sp}\varepsilon_{fspd}$ shall be substituted for f_{fbd}.

γ_b : Member factor, generally taken to be 1.3

(2) When the bending moment and the design capacity of member cross-sections are calculated according to the direction of section force for unit width of member sections or members, calculations shall be performed on the basis of assumptions (i) to (iii) given below.

(i) Fiber strain is proportional to the distance from the neutral axis.
(ii) Tensile stress of concrete is ignored.
(iii) The tensile force - strain curve of the CFRM follows **3.4.3**.

(3) For fiber rupture flexural failure, the capacity when any reinforcement reaches design ultimate strain ε_{fud} as shown in **Fig. 6.2.1** is taken to be the design capacity of the member cross-sections. The member factor γ_b may generally be taken as 1.15 to 1.3.

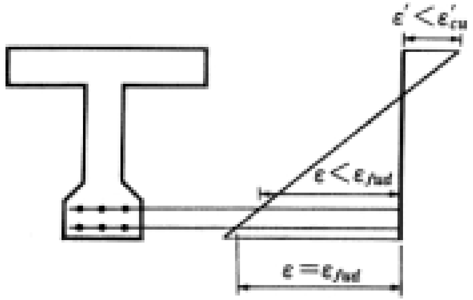


Fig. 6.2.1 Strain condition at fiber rupture flexural failure in members with multi-layer reinforcement

(4) For flexural compression failure, the compressive stress distribution in the concrete may be assumed to be identical to the rectangular compressive stress distribution (equivalent stress block) given in JSCE Standard Specification (Design), 6.2.1(3). The member factor γ_b may generally be taken as 1.3.

(5) The design capacity of a member cross-section subjected to combined biaxial bending moment and axial forces shall be calculated according to (2) to (4) explained above.

(6) When the effect of axial forces is negligible, the cross-sectional capacity may be calculated as for a flexural member. Axial forces may be taken to be negligible when $e/h \geq 10$, where h is section height and eccentricity e is the ratio of design flexural moment M_d to design axial compressive force N'_d.

[COMMENTS]:

Particularly when high ductility is required, measures such as combining CFRM with steel reinforcement, confinement of compression zone concrete etc., have to be implemented.

(1) As the compressive strength of CFRM is lower than the tensile strength and subject to significant variation, the effects of compressive strength are to be ignored for the purposes of calculation of axial compressive capacity N'_{oud}.

The effects of using CFRM for spiral reinforcement are allowed for in Eq (6.2.2). The design value ε_{fspd} for the strain of spiral reinforcement at ultimate limit state has been set at 2000×10^{-6}, allowing for the fact that in the equation for axial compression capacity when steel reinforcement is used, the steel is assumed to yield on the basis of test results. If the design strength when spiral reinforcement is regarded as a bent portion f_{fbd} is lower than $E_{sp}\varepsilon_{fspd}$, the latter may be substituted.

(3) As there is no yielding and no plastic region when CFRM is used, rupture begins from reinforcing materials when the strain of the reinforcement reaches the ultimate strain. The first rupturing of the reinforcing material is thus generally simultaneous with the ultimate state of the member, and capacity is calculated from the strain distribution obtained assuming plane sections remain plane. In a member with steel reinforcement arranged in multiple layers, stress may be evaluated from the position of the center of gravity of the steel, but for CFRM, as **Fig. 6.2.1** illustrates, fiber rupture flexural failure takes place when the outermost reinforcement reaches the ultimate strain. If different types of CFRM are

used within the same section, or if bonded and unbonded reinforcing material is used together, these circumstances must be allowed for in calculating the capacity.

(4) In flexural compression failure, it is possible to calculate capacity in the same way as for steel, therefore calculation of capacity using the equivalent stress block method is allowed here.

6.2.2 Structural detail

(1) Minimum axial reinforcement

(i) In concrete members reinforced with CFRM where axial forces are dominant, the quantity of axial reinforcement shall be not less than $0.8(E_0/E_{fu})$% of the calculated minimum cross-sectional area of the concrete, where E_0 is reference Young's modulus (=200 kN/mm^2), and E_{fu} is Young's modulus of axial reinforcement. The "calculated minimum cross-sectional area of the concrete" here refers to the minimum cross-sectional area of concrete required for axial support only.
Where the section is larger than the minimum required section, the amount of axial reinforcement should preferably be in excess of $0.1(E_0/E_{fu})$% of the concrete cross-sectional area.

(ii) The ratio of tensile reinforcement in beam members where the effects of bending moment are dominant shall generally be not less than $(35\ f_{tk}/f_{fuk})$% or 0.2%, whichever is the greater. For T-cross sections, the amount of axial tensile reinforcement shall be not less than 1.5 times as great as the above value, relative to the effective cross-sectional area of the concrete. In this, f_{tk} is the characteristic value of the tensile strength of the concrete, and f_{fuk} is the characteristic value of the tensile strength of the tensile reinforcement. The "effective cross-sectional area of the concrete" here refers to the effective depth of the section d multiplied by the web width b_w.

(2) Maximum axial reinforcement

In concrete members where axial forces are dominant, the amount of axial reinforcement shall generally be not greater than $6(E_0/E_{fu})$% of the cross-sectional area of the concrete.

[COMMENTS]:
(1)
(i) The compressive strength of CFRM can be ignored for the purpose of calculating axial compressive capacity, but in order to ensure axial rigidity, a minimum amount of axial reinforcement has been specified, as for steel reinforcement. Where the member cross section is larger than the calculated minimum cross-sectional area of the concrete, while a minimum axial reinforcement is required from the point of view of cracking, as CFRM is not liable to corrosion, the requirements given here have been relaxed slightly as compared to those for steel reinforcement. Where CFRM is used in conjunction with steel, however, the value of (steel quantity + (E_{fu}/E_0) · CFRM quantity) must be not less than 0.15% of the cross-sectional area of the concrete.
(ii) Where the ratio of tensile reinforcement is extremely low, the reinforcement ruptures as soon as cracking appears, inducing a state of brittle failure. The minimum amount of reinforcement is prescribed in order to avoid this. Allowing for the size effect of the member, the minimum tensile reinforcement ratio may be either $(35\ k_1 f_{tk}/f_{fuk})$% or 0.2%, whichever is the greater. k_1 is obtained from Eq. (C 6.2.1).

$$k_1 = 0.6 / (h^{1/3}) \quad \text{(C 6.2.1)}$$

where h is total member depth (m), provided that $0.4 \leq k_1 \leq 1.0$.

6.3 SAFETY VERIFICATION OF SHEAR FORCES

6.3.1 General

It shall be in accordance with JSCE Standard Specifications (Design), 6.3.1.

6.3.2 Design shear forces of beam members

It shall be in accordance with JSCE Standard Specifications (Design), 6.3.2.

6.3.3 Design shear capacity of beam members

(1) Design shear capacity V_{ud} is obtained from Eq. (6.3.1), provided that when bent-up reinforcement and stirrups are used together for shear reinforcement, the stirrups bear not less than 50% of shear force required to be borne by the shear reinforcement.

$$V_{ud} = V_{cd} + V_{sd} + V_{ped} \quad (6.3.1)$$

where

V_{cd} : design shear capacity of beam members not used in shear reinforcement, obtained from Eq. (6.3.2).

$$V_{cd} = \beta_d \cdot \beta_p \cdot \beta_n \cdot f_{vcd} \cdot b_w \cdot d / \gamma_b \quad (6.3.2)$$

where

$$f_{vcd} = 0.2\sqrt[3]{f'_{cd}} \text{ (N/mm}^2\text{), provided that } f_{vcd} \leq 0.72 \text{ N/mm}^2 \quad (6.3.3)$$

$\beta_d = \sqrt[4]{1/d}$ (d:m); if $\beta_d > 1.5$ then $\beta_d = 1.5$

$\beta_p = \sqrt[3]{100 p_w E_{fu} / E_0}$; if $\beta_p > 1.5$ then $\beta_p = 1.5$

$\beta_n = 1 + M_0/M_d$; (if $N'_d \geq 0$); if $\beta_n > 2$ then $\beta_n = 2$

$1 + 2\, M_0/M_d$ (if $N'_d < 0$); if $\beta_n < 0$ then $\beta_n = 0$

N'_d : design axial compressive force

M_d : design bending moment

M_0 : bending moment required to cancel out stresses set up by axial forces in the tensioned edge, relative to design bending moment M_d

E_{fu} : Young's modulus of tensile reinforcement

E_0 : reference Young's modulus (=200 kN/mm^2)

b_w : width of web

d : effective depth

$p_w = A_f/(b_w d)$

A_f : cross-sectional area of tensile reinforcement

f'_{cd} : design compressive strength of concrete, in units of N/mm^2

γ_b : generally = 1.3

V_{sd} : design shear capacity borne by shear reinforcement, obtained from Eq. (6.3.4)

$$V_{sd} = \left[A_w E_w \varepsilon_{fwd} (\sin\alpha_s + \cos\alpha_s) / s_s + A_{pw} \sigma_{pw} (\sin\alpha_p + \cos\alpha_p) / s_p \right] z / \gamma_b \qquad (6.3.4)$$

A_w : total cross-sectional area of shear reinforcement in section s_s

E_w : Young's modulus of shear reinforcement (= E_{fu})

ε_{fwd} : design value of shear reinforcement strain in ultimate limit state, obtained from Eq. (6.3.5). Where $E_w\varepsilon_{fwd}$ is greater than the design value for the strength of the bent portion f_{fbd}, f_{fbd} is substituted for $E_w\varepsilon_{fwd}$. f_{fbd} may be obtained from Eq. (3.4.1).

$$\varepsilon_{fwd} = \sqrt{f'_{mcd} \frac{p_w E_{fu}}{p_{web} E_w}} \left[1 + 2\left(\frac{\sigma'_N}{f'_{mcd}} \right) \right] \times 10^{-4} \qquad (6.3.5)$$

α_s : angle formed by shear reinforcement and member axis

s_s : spacing of shear reinforcement

p_{web} : $A_w/(b_w \cdot s_s)$

A_{pw} : total cross-sectional area of shear reinforcement tendons in section s_p

σ_{pw} : effective tensile stress of shear reinforcement

$\sigma_{pw} = \sigma_{wpe} + E_{fpw}\varepsilon_{fwd} \le f_{fpud}$

σ_{wpe} : effective tensile stress of shear reinforcement tendons

E_{fpw} : Young's modulus of shear reinforcement

f_{fpud} : design tensile strength of shear reinforcement

α_p : angle formed by shear reinforcement and member axis

s_p : spacing of shear reinforcement

z : distance from point of action of compressive stress resultant force, generally d / 1.15

σ'_N : average axial compressive stress

$\sigma'_N = (N'_d + P_{ed}) / A_g$

if $\sigma'_N > 0.4f'_{mcd}$ then $\sigma'_N = 0.4f'_{mcd}$

P_{ed} : effective tensile force in axial tendons

A_g : total cross-sectional area

f'_{mcd} : design compressive strength of concrete allowing for size effect (N/mm^2)

$$f'_{mcd} = \left(\frac{h}{0.3} \right)^{-1/10} \cdot f'_{cd}$$

f'_{cd} : design compressive strength of concrete, in N/mm^2

h : member depth (m)

γ_b : generally = 1.15

V_{ped} : component of effective tensile force of axial tendons parallel to shear force, obtained from Eq. (6.3.6)

$$V_{ped} = P_{ed} \sin\alpha_p / \gamma_b \qquad (6.3.6)$$

α_p : angle formed by shear reinforcement and member axis

γ_b : generally = 1.15

(2) When beam members are supported directly, V_{ud} need not be investigated for the zone from the support face to one-half of the depth h of the members. In this zone, shear reinforcement more than the

minimum required shall be placed in the cross section from the support face to $h/2$. In members of non-uniform section, the depth at the support face may be adopted as the member depth; parts of a haunch where the gradient is less than 1:3 shall be considered to be effective.

(3) The design diagonal compressive capacity V_{wcd} of web concrete to shear force shall be obtained from Eq. (6.3.7).

$$V_{wcd} = f_{wcd} \cdot b_w \cdot d / \gamma_b \tag{6.3.7}$$

where

$f_{wcd} = 1.25\sqrt{f'_{cd}}$ (N/mm^2); provided that $f_{wcd} \leq$ 7.8 N/mm^2

γ_b = generally = 1.3

(4) Web width of members

(i) Where the diameter of a single duct in prestressed concrete members is equal to or greater than 1/8 of the web width, the web width assumed in Eq. (6.3.2) shall be smaller than the actual web width b_w. In such a case, the web width may generally be reduced by the total of the diameters of the ducts ϕ arranged in that section, giving b_w - $1/2\Sigma\phi$.

(ii) For members with web widths varying in the direction of member depth, other than those with circular sections, the minimum width b_w within the range of effective depth d shall be adopted. For members with multiple webs, b_w shall be the total width of all webs. For solid or hollow circular sections, web width b_w shall be either the length of one side of a square with the equivalent area, or as the total width of webs of square boxes having the same area. In these cases, the area of axial tensile reinforcement A_f shall be the cross-sectional area of reinforcement in 1/4 (90°) of the cross section of the tensioned side. The effective depth d shall be either the distance from the compression edge of the square or box of equivalent area to the centroid of the reinforcement, accounted for as A_f. These definitions of axial tensile reinforcement area shall not apply in calculation of flexural capacity.

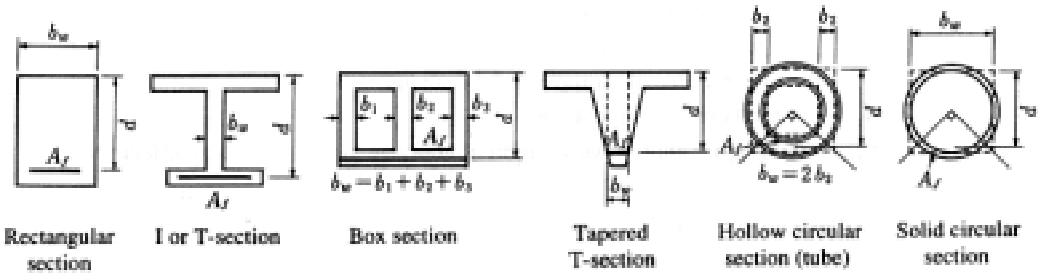


Fig. 6.3.1 Definitions of b_w and d for various cross-sections

[COMMENTS]:

(1) The design shear force V_{ud}, as shown in Eq. (6.3.1), is given as the sum of the components carried by the concrete V_{cd} and by the shear reinforcement V_{sd}, except that the components (V_{ped}) of effective tensile force in the axial reinforcement parallel to the shear force is ignored.

Previous studies indicate that the shear capacity of beam members with CFRM used for tensile reinforcement but without shear reinforcement can generally be evaluated by taking into account the axial rigidity of the tensile reinforcement. V_{cd} is thus calculated according to the equation used for steel, allowing for the ratio of the Young's modulus of CFRM to that of steel.

The strain ε_{fwd} of shear reinforcement at the ultimate limit state is affected by concrete strength, the rigidity of tensile and shear reinforcement, and axial compression force. These functions are given by Eq. (6.3.5). Eq. (6.3.5) is derived from the most recent findings of research on the design shear capacity of beam members using CFRM, shown below. These findings offer a more accurate method than the conventional one for estimating shear stress, by incorporating a more realistic shear resistance mechanism. This method may be followed in estimating the ultimate shear capacity.

The shear capacity obtained by the method given below is generally greater than that obtained from Eq. (6.3.1). The method below is greatly simplified, for instance by conservatively ignoring the effect of the shear span-to-depth ratio on shear capacity, but in some instances it will give a lower shear capacity than Eq. (6.3.1), for example when the main reinforcement has high rigidity.

Design shear capacity when shear reinforcement does not break is calculated as follows:

$$V_{ud} = V_{cd} + V_{sd} \tag{C 6.3.1}$$

where

V_{cd} = design shear force carried by concrete, obtained from Eq. (C 6.3.2)

$$V_{cd} = V_{czd} + V_{aid} \tag{C 6.3.2}$$

where

V_{czd} : design shear force carried by concrete in compression zone, obtained from Eq. (C 6.3.3)

$$V_{czd} = \beta f'_{mcd}\, x_e b_w / \gamma_b \tag{C 6.3.3}$$

V_{aid} : design shear force carried by concrete in diagonal cracking zone, obtained from Eq. (C 6.3.4)

$$V_{aid} = \beta_P \beta_{pE} f'^{1/3}_{mcd} (h - x_e) b_w / \gamma_b \tag{C 6.3.4}$$

V_{sd} = shear capacity carried by shear reinforcement, obtained from Eq. (C 6.3.5)

$$V_{sd} = A_w E_w \varepsilon_{fwd} (h - x_e) b_w / (\tan\theta_{cr} s_s) / \gamma_b \tag{C 6.3.5}$$

x_e : depth of concrete compression zone at ultimate, obtained from Eq. (C 6.3.6)

$$x_e = [1 - 0.8 (p_{web} E_{fw})^{-0.2}\left[1 + \left(\frac{\sigma'_N}{f'_{mcd}}\right)^{0.7}\right] x \tag{C 6.3.6}$$

ε_{fwd} : strain in shear reinforcement at ultimate limit state, obtained from Eq. (C 6.3.7)

$$\varepsilon_{fwd} = 0.0001 \sqrt{f'_{mcd} \frac{p_w E_{fu}}{p_{web} E_w}} \left[1 + 2\left(\frac{\sigma'_N}{f'_{mcd}}\right)\right] \tag{C 6.3.7}$$

θ_{cr} : angle of diagonal cracking, obtained from Eq. (C 6.3.8)

$$\theta_{cr} = 45\left[1 - \left(\frac{\sigma'_N}{f'_{mcd}}\right)^{0.7}\right] \tag{C 6.3.8}$$

$$\beta = 0.2\left(\frac{\sigma'_N}{f'_{mcd}}\right)^{0.7}$$

$\beta_P = 1 - 5\dfrac{\sigma'_N}{f'_{mcd}}$; if $\beta_P < 0$ then $\beta_P = 0$

$\beta_{pE} = 0.24\left(\dfrac{p_w E_{fu} + 10 p_{web} E_w}{5000k} + 0.66\right)$; if $\beta_{pE} > 0.40$ then $\beta_{pE} = 0.40$

$$k = 1 - \left(\frac{\sigma'_N}{f'_{mcd}}\right)^{0.1}$$

f'_{mcd} : design compressive strength of concrete, allowing for size effect (N/mm^2)

$$f'_{mcd} = \left(\frac{h}{0.3}\right)^{-1/10} \cdot f'_{cd}$$

f'_{cd} : design compressive strength of concrete (N/mm^2)
b_w : web width
d : effective depth
h : beam height (m)
A_f : cross-sectional area of tension reinforcement (mm^2)
A_w : total cross-sectional area of shear reinforcement in zone s_s
$p_w = A_f/(b_w d)$
$p_{web} = A_w/(b_w s_s)$
E_{fu} : Young's modulus of tension reinforcement (N/mm^2)
E_w : Young's modulus of shear reinforcement (N/mm^2)
$\sigma'_N = (N'_d + P_{ed})/A_g$ (N/mm^2); if $\sigma'_N > 0.4 f'_{mcd}$ then $\sigma'_N = 0.4 f'_{mcd}$
N'_d : design axial compression force
P_{ed} : effective tensile force of axial reinforcement
A_g : cross-sectional area of entire section
s_s : spacing of shear reinforcement
x : position of neutral axis according to elastic theory, ignoring tension section
γ_b : generally = 1.3

Design shear capacity when shear reinforcement breaks by fiber rupture is calculated as follows:

$$V_{ud} = V_{c0} - \beta_m(V_{c0} - V_{czd}) + \beta_m V_{aid} + \beta_m V_{sd} \qquad \text{(C 6.3.9)}$$

where

V_{c0} : load at which diagonal cracking occurs, obtained from Eq. (C 6.3.10)

$$V_{c0} = \beta_0 \beta_d f'_{cd} x_0 b_w/\gamma_b + \beta_{P0}\beta_{pE0}\beta_d f'^{1/3}_{cd}(h-x_0)b_w/\gamma_b \qquad \text{(C 6.3.10)}$$

V_{czd} : design shear force carried by concrete in compression zone; may be obtained from Eq. (C 6.3.3)
V_{aid} : design shear force carried by concrete in diagonal cracking zone; may be obtained from Eq. (C 6.3.4)
V_{sd} : design shear force carried by shear reinforcement; may be obtained from Eq. (C 6.3.5)
x_0 : depth of compression zone in concrete at onset of diagonal cracking, obtained from Eq. (C 6.3.11)

$$x_0 = \left[1 + \left(\frac{\sigma'_N}{f'_{cd}}\right)^{0.7}\right] x \qquad \text{(C 6.3.11)}$$

$$\beta_0 = 0.14\left(\frac{\sigma'_N}{f'_{cd}}\right)^{0.7}$$

$\beta_d = \sqrt[4]{1000/d}$; if $\beta_d > 1.5$ then $\beta_d = 1.5$

$\beta_{P0} = 1 - 5\dfrac{\sigma'_N}{f'_{cd}}$; if $\beta_{P0} < 0$ then $\beta_{P0} = 0$

$$\beta_{pE0} = 0.17\left(\frac{p_w E_{fu}}{5000k} + 0.66\right)$$; if $\beta_{pE0} > 0.28$ then $\beta_{pE0} = 0.28$

$$k = 1 - \left(\frac{\sigma'_N}{f'_{cd}}\right)^{0.7}$$

$$\beta_m = \frac{f_{ud}}{E_w \varepsilon_{fud}}$$

f_{ud} : design tensile strength of shear reinforcement, taken as equivalent to design strength of bent portion f_{fbd}, where f_{fbd} may be obtained from Eq. (3.4.1)

Design shear capacity V_{ud}, as shown in Eq. (C 6.3.1), is expressed as the sum of the components carried by concrete V_{cd} and by shear reinforcement V_{sd}. For each of these components, the effects of the Young's modulus of the tendons are evaluated as the rigidity obtained by multiplying the reinforcement ratio by the Young's modulus of the reinforcing material.

The shear force carried by the concrete in the compression zone increases as the axial compression force increases. This is expressed by Eq. (C 6.3.3).

The mode of failure of the beam varies depending on the rigidity (reinforcement ratio × Young's modulus) of the main reinforcement and the shear reinforcement. That is, as the rigidity of the main reinforcement and the shear reinforcement increases, the failure mode shifts from diagonal tensile failure to shear compressive failure. β_{pE} in Eq. (C 6.3.4) signifies that when the rigidity of the main reinforcement and the shear reinforcement is low and the beam undergoes diagonal tensile failure, the shear transmission stress of the diagonal cracking zone increases as the rigidity of the main reinforcement and the shear reinforcement increases. However, when the rigidity is high and the beam undergoes shear compressive failure, the shear transmission force of the concrete in the diagonal cracking zone remains constant regardless of the rigidity of the main reinforcement and the shear reinforcement (**Fig. C 6.3.2**).

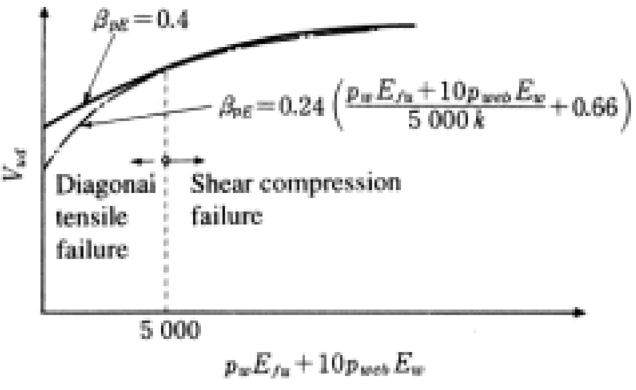


Fig. C 6.3.2 Effect of rigidity of longitudinal and shear reinforcement on shear strength

The shear span-to-depth ratio also affects the mode of failure, although previous studies have confirmed that at shear span-to-depth ratios of 2 or more, if the reinforcement has low rigidity, diagonal tensile failure will occur. Where axial compressive force is present, the mode of failure shifts from diagonal tensile failure to shear compressive failure. Previous studies have confirmed that shear compressive failure occurs even at low reinforcement rigidity, and the term k in β_{pE} (Eq. C 6.3.4) is included to allow for this effect. That is, the reference value ($p_w E_{fu}$ + 10$p_{web} E_w$ = 5000) for the case where axial compressive force is not acting, decreases as the axial compressive force increases.

The angle of diagonal cracking, i.e. the angle of the truss diagonals, becomes shallower as the axial compressive force increases. This is expressed in Eq. (C 6.3.8).

Shear reinforcement is thought to fail if the stress in shear reinforcement at ultimate limit state $E_w \varepsilon_{fwd}$ is greater than the strength of the bent portion f_{fbd}, obtained from Eq. (3.4.1). In this case, the design shear capacity V_{ud} is obtained from Eq. (C 6.3.9). That is, stress in the shear reinforcement after the onset of diagonal cracking, and components V_{czd} and V_{aid}, are thought to vary linearly according to the acting shear force, and components V_{czd}, V_{aid} and V_{sd} are reduced by a factor β_m, obtained by dividing the failure strength of the shear reinforcement by the shear reinforcement stress $E_w \varepsilon_{fud}$, calculated assuming non-failure of the shear reinforcement (**Fig. C 6.3.3**).

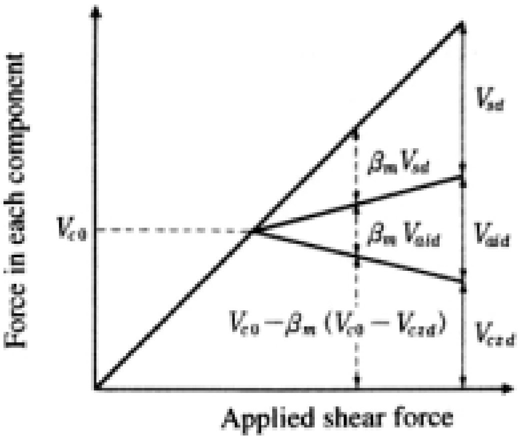


Fig. C 6.3.3 Modeling of each component of shear capacity

The method given here for calculation of shear capacity is derived from dynamic models agreeing with empirical facts, such as that the angle of the main compressive stress within the concrete is not 45° even if the angle of shear cracking within the shear span is generally 45° relative to the member axis, and that the load stress of the concrete carried outside of the truss mechanism varies with the acting shear force, and its value is not equivalent to the shear capacity of members without shear reinforcement. Eq. (C 6.3.5) which follows this method gives the shear force carried by shear reinforcement straddling diagonal cracks; where axial forces are not present, the angle of diagonal cracking is 45°, and the expression approximates the equation given in the JSCE Standard Specification, and also Eq. (6.3.4) of the present Recommendation. The difference between the two equations is that Eq. (C 6.3.5) incorporates a term (h-x_e) expressing the depth of the diagonal cracking zone, whereas Eq. (6.3.4) incorporates a term z expressing the arm length of the truss. According to the model referred to above, shear forces other than those carried by the truss mechanism are expressed by V_{czd} in Eq. (C 6.3.3), and this value generally varies with the acting shear force (cf. **Fig. C 6.3.3**). The sum of this term V_{czd} and V_{aid}, the shear force transmitted by the interlocking of the aggregate in the diagonal cracking zone etc. (cf. Eq. (C 6.3.4)), is generally constant, corresponding closely with Eq. (6.3.2).

(3) The width of diagonal cracking is thought to be wider when CFRM is used than when steel reinforcement is used. The compressive capacity and rigidity of concrete where cracking is present

decreases as the strain perpendicular to the cracks increases, therefore diagonal compressive failure capacity is thought to be lower than when steel reinforcement is used. This hypothesis is yet to be confirmed experimentally, however, and in the present specifications, diagonal compressive capacity of reinforced concrete beams is evaluated conservatively in Eq. (6.3.7).

6.3.4 Design punching shear capacity of planar members

(1) When the loaded area is positioned far from free edges or openings, and the eccentricity of the load is small, the design punching shear capacity V_{pcd} may be determined by Eq. (6.3.8).

$$V_{pcd} = \beta_d \cdot \beta_p \cdot \beta_r \cdot f_{pcd} \cdot u_p d / \gamma_b \qquad (6.3.8)$$

where

$$f_{pcd} = 0.2\sqrt{f'_{cd}} \ \text{(N/mm}^2\text{)}; \ f_{pcd} \text{ shall be} \leq 1.2 \text{ N/mm}^2 \qquad (6.3.9)$$

$\beta_d = \sqrt[4]{1/d}$ (d:m); if $\beta_d > 1.5$ then $\beta_d = 1.5$

$\beta_p = \sqrt[3]{100 p E_{fu} / E_0}$; if $\beta_p > 1.5$ then $\beta_p = 1.5$

$\beta_r = 1 + 1/1(1+0.25\ u/d)$

f'_{cd} : design compressive strength of concrete (N/mm^2)

u : peripheral length of loaded area

E_{fu} : Young's modulus of tensile reinforcement

E_0 : standard Young's modulus (=200 kN/mm^2)

u_p : peripheral length of the design cross-section at d/2 from the loaded area

d, p : effective depth and reinforcement ratio, defined as the average values for the reinforcement in both directions.

γ_b : generally = 1.3

(2) When the loaded area is located in the vicinity of free edges or openings in members, the reduction of the punching shear capacity shall be allowed for.

(3) When loads are applied eccentrically to the loaded area, the effects of flexure and torsion shall be allowed for.

[COMMENT]:

(1) As with the shear capacity of beam members without shear reinforcement, the punching shear capacity may generally be evaluated by allowing for the axial rigidity of the reinforcement. The Young's modulus of the CFRM is therefore allowed for in the calculation of design punching shear capacity V_{pcd}.

6.3.5 Structural details

(1) In beam members, stirrups not less than 0.15(E_0/E_{fu})% shall; be arranged over the entire member length, where E_0 is standard Young's modulus (=200 kN/mm^2), and E_{fu} is Young's modulus of axial reinforcement. The spacing of the stirrups shall generally be not more than 1/2 of the effective depth of the member, and not more than 30 cm. This provision (1) need not be applied to planar members.

(2) Shear reinforcement equivalent to that required by calculation shall also be arranged in sections equivalent to the effective depth outside of the section where it is required.

(3) The ends of stirrups and bent bars shall be adequately embedded in the concrete on the compressive side.

[COMMENT]:

(1) When steel reinforcement is used, stirrups equivalent to not less than 0.15% of the concrete area are installed to prevent sudden failure due to the onset of diagonal cracking. Based on this provision, a minimum amount of stirrup of $0.15(E_0/E_{fu})$% is also imposed here for CFRM reinforcement. As most CFRM have low elasticity and small cross-sectional areas, the spacing requirements given here are slightly stricter than those for steel.

6.4 TORSION SAFETY

6.4.1 General

(1) For structural members not significantly influenced by torsional moment, and those subjected to compatibility torsional moment, the torsional safety studies given in section 6.4 may be omitted. "Structural members not significantly influenced by torsional moment" here refers to members in which the ratio of the design torsional moment M_{td} to the design pure torsional capacity M_{tcd}, calculated according to 6.4.2 (members without torsional reinforcement), multiplied by structural factor γ_i, is less than 0.2 for all sections.

(2) When the effects of design torsional reinforcement are not negligible, torsion reinforcement shall be arranged in accordance with 6.4.2.

6.4.2 Design torsional capacity

(1) Torsional capacity in members without torsional reinforcement shall be in accordance with "JSCE Standard Specification (Design)", section 6.4.2.

(2) Torsional capacity in members with torsional reinforcement shall be calculated according to appropriate methods.

[COMMENT]:

(2) Studies of CFRM used for torsional reinforcement have not yet been adequately carried out. Design torsional capacity in members with torsional reinforcement must therefore be investigated experimentally and analytically based on reliable techniques.

CHAPTER 7: SERVICEABILITY LIMIT STATES

7.1 GENERAL

It shall be in accordance with JSCE Standard Specification (Design), 7.1.

7.2 CALCULATION OF STRESS AND STRAIN

It shall be in accordance with JSCE Standard Specification (Design), 7.2, with the following assumptions made regarding CFRM:
(i) CFRM is elastic body;
(ii) The Young’s modulus of CFRM is determined according to **3.4.3(2)**.

7.3 STRESS LIMITATION

It shall be in accordance with JSCE Standard Specification (Design), 7.3. The limitation of tensile stress in CFRM shall be determined by testing, according to the type of reinforcing material used.

[COMMENT]:
Unlike reinforcing or prestressing steel, CFRM undergoes failure at less than their static strength when subjected to sustained stress for long periods (i.e. creep failure).

Creep failure strength is to be tested according to JSCE-E 533 "Test Method for Creep Failure of Continuous Fiber Reinforcing Materials" based on the test results up to 1000 hours, extrapolating the creep failure strength at 1 million hours. The limitation of tensile stress in CFRM may generally be derived by multiplying the characteristic value of creep failure strength f_{fck} by a reduction factor of 0.8, given that the creep failure strength varies significantly depending on the fiber type, and given also that creep testing requires long periods of time. The limit value shall be not more than 70% of the characteristic value for tensile strength.

Creep failure as a phenomenon properly belongs under investigation of ultimate limit state, although it is placed in this section on serviceability limit state owing to the nature of the loads studied. For this reason, a reduction factor is used instead of a material factor.

7.4 CRACKING

7.4.1 General

(1) It shall be examined by an appropriate method that cracking in concrete does not impair the function, durability, appearances of the structures.

(2) This clause shall be applied to the verification of cracking caused by flexural moment, shear force,

torsional moment and axial force.

(3) Where the appearances of the structure is deemed important, the crack width on the concrete surface shall generally be kept within an allowable crack width considered acceptable for aesthetic considerations. Verification of cracking may be omitted for structures with particularly short service life, temporary structures, or structures where aesthetic considerations are not important.

(4) Where watertightness is important, the verification of cracking shall be done according to JSCE Standard Specification (Design), 7.4.1(4).

[COMMENTS]:

(1) Unlike steel materials, CFRM is considered to be free from corrosion. Cracking in concrete structures, however, generally results in loss of watertightness, airtightness and other functions, deterioration of the concrete, excessive deformation, unattractive appearance etc. Cracking in concrete must therefore be examined according to appropriate methods, to ensure that the functions, appearances of the structure are not impaired.

(3), (4) Verification of serviceability limit state when the intended purpose of the structure dictate particular aesthetic requirements, watertightness and airtightness requirements shall if necessary be made on the basis of a maximum allowable crack width.

7.4.2 Allowable crack width

(1) The allowable flexural crack width w_a shall generally be determined based on the intended purpose of the structure, environmental conditions, member conditions etc.

(2) Allowable crack widths set for aesthetic considerations may generally be set to not more than 0.5 mm, depending on the ambient environment of the structure.

(3) Crack limitations and allowable crack widths set for considerations of watertightness shall be based on JSCE Standard Specification (Design), 7.4.2(3).

[COMMENTS]:

(1) Allowable crack widths must be determined based on the intended purpose of the structure - function, relative importance, service life etc., the ambient environment and loading conditions, and also on member conditions such as the effects of axial force, covering, variation in crack widths etc.

(2) As CFRM is generally considered to be non-corrosion, there is no necessity to set allowable crack widths out of consideration of corrosion. Excessive crack width, however, would impair the appearance of the structure, as well as having a negative psychological impact. Whether or not cracking is likely to occur should first be investigated, and if cracking to be allowed, an appropriate allowable cracking width should be set based on aesthetic considerations, depending on the type of structure, the distance of the structure from the eyes of the casual onlooker, etc. Generally speaking, where main reinforcement is not prestressed, if the CFRM has low rigidity, large crack width may occur even at low load levels.

Where CFRM is used in conjunction with steel reinforcement, steel corrosion must also be considered in setting the allowable crack width, and in this case the allowable crack width is based on JSCE Standard Specification (Design), 7.4.2. Where steel reinforcement is not used, the maximum allowable crack width for members in public view has been set at not more than 0.5 mm.

7.4.3 Verification of flexural cracks

(1) Verification of flexural cracks may be omitted where the tensile stress of the concrete due to flexural moment and axial forces is lower than the design tensile strength of the concrete considering size effect.

(2) In the verification of flexural cracks shall be made, in general, the crack width w obtained from Eq. (7.4.1) shall be confirmed to be less than the allowable crack width w_a.

$$w = k\{4c + 0.7(c_f - \phi)\}(\sigma_{fe}/E_f\,(or\,\sigma_{pe}/E_{fp}) + \varepsilon'_{csd}) \qquad (7.4.1)$$

where

k = constant expressing the effects of bond characteristics and multiple placement of reinforcing materials; generally 1.0~1.3

c = concrete cover (mm)

c_f = center-to-center distance between reinforcing materials (mm)

ϕ = diameter of reinforcing materials (mm)

ε'_{csd} = compressive strain for evaluation of increment of crack width due to shrinkage and creep of concrete

σ'_{fe} = stress increase in reinforcement

E_f = Young's modulus of reinforcement

σ_{fpe} = stress increase in tendons

E_{fp} = Young's modulus of tendons

(3) The reinforcement and tendons to be examined for flexural cracks shall generally be the tensile reinforcement nearest to the concrete surface. Stress and strain shall be obtained according to section **7.2** above.

[COMMENTS]:

(1) Design tensile strength of concrete considering the size effect shall be according to Eq. (C 7.4.1) in the JSCE Standard Specification (Design).

(2) Eq. (7.4.1) is the same as that used for calculation of crack widths in concrete members using conventional steel reinforcement. The width and spacing of flexural cracks is generally affected significantly by the bond between the reinforcement and the concrete. CFRM may be classified according to their method of manufacture and surface geometry as strand, braid, wound, machined, lattice etc., and each type is considered to have different bond characteristics. Previous studies have found that when the surface is treated to give bond characteristics similar to conventional deformed steel bars, the spacing of cracks in concrete members is almost identical to that when deformed steel bars are used. In cases such as this, crack width can be calculated according to Eq. (7.4.1). The bond properties of CFRM are generally between those of round steel bars and deformed steel bars. The value of k in Eq. (7.4.1) must therefore be determined appropriately for each CFRM type, although for CFRM which has

been confirmed to have bond characteristics similar to those of deformed steel bars, a value of k = 1.0 may be adopted.

The term ε'_{csd} in Eq. (7.4.1) expresses the effects of concrete shrinkage and creep on crack widths, and must be determined on the basis of the surface configuration of the member, ambient environment, stress levels etc. Little basic data is available regarding ε'_{csd}, and further research in this area is required, but on the basis of an overall consideration of existing crack width formulae etc., ε'_{csd} can generally be taken to be = 150×10^{-6}.

When latticed CFRM is used, the lattice spacing also affects crack spacing; this effect is allowed for by calculating crack spacing l_k, calculating the crack width according to the following eq.:

$$w = l_k (\sigma_{fe} / E_f + \varepsilon'_{csd}) \qquad \text{(C 7.4.1)}$$

The basic policy regarding control of crack widths is to keep the width of cracks on the concrete surface below the allowable crack width determined on the basis of structural conditions and the concrete cover, although for convenience of design, for normal members a limit is set on the increase of strain in the CFRM due to permanent loads, considered to have minimal effect on crack widths; this provision allows the verification of crack widths in (2) to be omitted. Generally speaking, if either the strain increase in the reinforcement due to permanent loads σ_{fp}/E_f, or the strain increase in the tendons σ_{fpp}/E_{fp}, is less than 500×10^{-6}, verification of crack width may be omitted.

(3) If CFRM is arranged in multiple layers, normally the stress used will be that of the tensile reinforcement closest to the concrete surface, although the effects on crack width of CFRM further inside the section may also be evaluated, if such effects have been determined experimentally to be present.

7.4.4 Verification of shear cracks

It shall be in accordance with JSCE Standard Specification (Design), 7.4.5.

[COMMENT]: Verification of shear cracks is normally to be done according to JSCE Standard Specification (Design), 7.4.5, although this verification may be omitted where the strain increase in the shear reinforcement due to permanent loads is less than 500×10^{-6}.

7.4.5 Verification of torsion cracks

It shall be in accordance with JSCE Standard Specification (Design), 7.4.6.

[COMMENT]: Verification of torsion cracks is normally to be done according to JSCE Standard Specification (Design), 7.4.6, although this verification may be omitted where the strain increase in the torsional reinforcement due to permanent loads is less than 500×10^{-6}.

7.4.6 Structural Details

It shall be in accordance with JSCE Standard Specification (Design), 7.4.7.

7.5 DISPLACEMENT AND DEFORMATION

7.5.1 General

It shall be in accordance with JSCE Standard Specification (Design), 7.5.1.

7.5.2 Allowable displacement and deformation

It shall be in accordance with JSCE Standard Specification (Design), 7.5.2.

7.5.3 Verification of displacement and deformation

It shall be in accordance with JSCE Standard Specification (Design), 7.5.3.

[COMMENT]: Verification of displacement and deformation is normally to be done according to JSCE Standard Specification (Design), 7.5.3, although where the Young's modulus of the CFRM is extremely low compared to the steel reinforcement, and where the quantity of reinforcement is low, the deformation will be greater than in conventional steel reinforced concrete members. The increased deformation makes shear cracking more likely, and this in turn is considered to affect the displacement and deformation of the whole structure. In cases where shear cracking occurs, it must be properly allowed for in calculating deformation levels.

7.6 VIBRATION

It shall be in accordance with JSCE Standard Specification (Design), 7.6.

CHAPTER 8: FATIGUE

8.1 GENERAL

It shall be in accordance with JSCE Standard Specification (Design), 8.1.

[COMMENT]: As with steel reinforcement, CFRM requires verification of fatigue sustained due to repeated tensile stress. Unlike steel, however, CFRM shows significant loss of strength due to secondary stress, and this point must be properly allowed for in examining the fatigue limit state. Tensile or shear reinforcement at intersections with shear cracks is more liable to undergo fatigue failure. Loss of fatigue strength in tensile reinforcement is not normally considered in conventional steel reinforced concrete members, but it must be allowed for in CFRM.

8.2 VERIFICATION OF FATIGUE

It shall be in accordance with JSCE Standard Specification (Design), 8.2.

[COMMENT]: As with steel reinforcement, verification relating to fatigue in CFRM is done by comparing design fatigue strength and design variable stress.

8.3 DESIGN VARIABLE SECTION FORCE AND EQUIVALENT NUMBER OF CYCLES

It shall be in accordance with JSCE Standard Specification (Design), 8.3.

[COMMENT]: Miner's hypothesis is thought to be applicable to CFRM as to steel, therefore the number of cycles equivalent to the design variable section force may be calculated in the same manner as for steel. In this case, though, the *S-N* curve for the fatigue strength of the CFRM is needed.

8.4 CALCULATION OF STRESS DUE TO VARIABLE LOAD

(1) Tensile stress in CFRM used for tensile reinforcement may be calculated according to section **7.2**.

(2) Stress in CFRM used for shear reinforcement may be calculated following JSCE Standard Specification (Design), 8.4(3) for steel shear reinforcement. The shear capacity V_{cd} of concrete without shear reinforcement shall be calculated according to Eq. (6.3.2) of the present recommendation.

(3) Stress of steel and concrete shall be calculated according to JSCE Standard Specification (Design), 8.4.

[COMMENTS]: (1), (2) Stress in CFRM may be calculated in the same way as for steel reinforcement, although the shear capacity V_{cd} of concrete without shear reinforcement, which is required for the calculation of stress in shear reinforcement, must be calculated according to Eq. (6.3.2) of the present recommendation, as the calculations differ from those for steel.

8.5 DESIGN SHEAR FATIGUE CAPACITY OF MEMBERS WITHOUT SHEAR REINFORCEMENT

Design shear fatigue capacity of flexurally reinforced members without shear reinforcement may be calculated following the provisions for steel reinforced concrete members given in JSCE Standard Specification (Design), 8.5, where V_{cd} and V_{pcd} shall be calculated according to Eqs. (6.3.2) and (6.3.8) of the present recommendation respectively.

[COMMENT]: Design shear fatigue capacity of members without shear reinforcement may be calculated as for steel reinforced members, although the static shear capacity for these calculations when applied to CFRM must be obtained from the equations given in the present recommendations.

CHAPTER 9: SEISMIC DESIGN

9.1 GENERAL

It shall be in accordance with JSCE Standard Specification (Seismic Design).

[COMMENT]: The provisions given in JSCE Standard Specification (Seismic Design) may be applied, although as CFRM generally do not yield, when they are used for flexural reinforcement the deformation after flexural yielding exhibited by steel reinforced concrete cannot be relied on.

CHAPTER 10: GENERAL STRUCTURAL DETAILS

10.1 GENERAL

It shall be in accordance with JSCE Standard Specification (Design), 9.1, where "steel" shall be taken to signify "steel or CFRM".

10.2 CONCRETE COVER

(1) Concrete cover shall be determined taking into consideration the quality of concrete, bar diameters, environmental conditions, errors in construction, and the importance of the structure.

(2) The minimum concrete cover shall be obtained from Eq. (10.2.1), and shall be not less than the bar diameter.

$$c_{\min} = \alpha \cdot c_0 \qquad (10.2.1)$$

where

c_{min} = minimum cover

α = cover factor dependent on design strength of concrete f'_{ck}, as follows:

$f'_{ck} \le 18$ N/mm^2: $\alpha = 1.2$

18 N/mm^2 < $f'_{ck} \le 34$ N/mm^2: $\alpha = 1.0$

34 N/mm^2 < f'_{ck}: $\alpha = 0.8$

c_0 = basic concrete cover, dependent on member as shown in **Table 10.2.1**

Table 10.2.1: Values of c_0 (basic concrete cover; mm)

Member	*Slab*	*Beam*	*Column*
c_0	25	30	35

(3) Concrete placed directly in the earth for footings or important members of structures, concrete cover should be not less than 75 mm.

(4) Concrete cover for concrete placed under water should be not less than 75 mm.

(5) Where concrete is vulnerable to abrasion by running water or similar, concrete cover should be increased as appropriate.

(6) Members placed in an acid river or subjected to severe chemical action shall be appropriately protected.

(7) Concrete cover in structures requiring special fire protection shall be determined taking into consideration the heat resistance of the CFRM, fire temperature and duration, aggregate characteristics etc.

[COMMENTS]:

(1) Adequate concrete cover of CFRM is necessary to realize full bond strength with the CFRM, to prevent deterioration of the CFRM, and to protect the CFRM in fires. Concrete cover should therefore be determined based on the designer's experience, taking into account the quality of the concrete, the characteristics and diameter of the CFRM, the effects of harmful substances acting on the concrete surface, the dimensions of the member, construction errors, the importance of the structure and so forth.

(2) Eq. (10.2.1) gives the minimum concrete cover. CFRM is generally highly resistant to corrosion, therefore there is no need to make special allowance for environmental conditions in **table 10.2.1**. Where CFRM is arranged in bundles, the diameter of the reinforcement shall be deemed to be that of a single rod of cross-sectional area equivalent to the sum of the cross-sectional areas of the individual strands in the bundle.

(3) This value may be reduced by a further 25 mm, provided the quality of cover is adequately assured by, for example, the use of high fluidity concrete.

(4) Concrete placed under water cannot be adequately compacted, the concrete sometimes does not adequately fill narrow spaces between the CFRM and the formwork, and the quality of underwater concrete is hard to determine, therefore a safe minimum of 75 mm has been set. For cast-in-place concrete piles etc., cover should be around 125 mm to allow for the presence of casings, irregularity of the inner face of drilled earth, installation of cages etc. All of these values are reduced by 25 mm from those given for steel reinforcement, in consideration of the superior corrosion resistance of CFRM which allows underwater environments to be treated as standard environments.

(5) Where concrete is vulnerable to abrasion, for instance on the upper side of a slab without effective protection, concrete cover should be increased by at least 10 mm, giving a section larger than the minimum required according to bearing capacity calculations.

(6) Members placed in acid rivers or exposed to strong chemical action should be provided with extra corrosion protection, as deterioration of the concrete cover cannot be prevented.

(7) A "structure requiring special fire protection" refers here to a structure showing little or no damage or weakness during a fire. Tests have found that the fire resistance of CFRM varies greatly from type to type, and the fire resistance of the proposed CFRM must be allowed for in determining concrete cover. If necessary the sue of additional fire-proofing layers etc. should be considered.

10.3 CLEAR DISTANCE

It shall be in accordance with JSCE Standard Specification (Design), 9.3, where "steel" shall be taken to signify "steel or CFRM".

10.4 BENT CONFIGURATIONS OF REINFORCEMENT

10.4.1 General

(1) CFRM may be placed bent within their elastic limit. The effects of elastic bending stress shall be allowed for in design.

(2) When bent CFRM is used, the design strength of the bent section shall be allowed for.

[COMMENT]: (2) The design strength of bent sections of CFRM is obtained from **3.4.1(3)** or **(4)**.

10.4.2 Stirrups, ties and hoops

(1) CFRM may be bent in closed, spiral, grid or solid configurations for use as stirrups, ties or hoops.

(2) The standard inside radius of bent sections of stirrups and hoops shall be 2ø, where ø = bar diameter.

[COMMENTS]:
(1) Ties and hoops serve to prevent buckling of axial reinforcement while constraining the inner concrete. They must therefore be closely spaced to ensure adequate effectiveness, and the ties and hoops themselves must be properly anchored. For this reason, the use of closed configurations is advised. Whichever configuration is used, the strength of bent sections and the panel points must be allowed for.

(2) The inside radius of bent sections of stirrups and hoops should be small as possible, from the practical point of view of containing the reinforcement, but making the inside radius too small could result in significant loss of strength.

10.4.3 Other reinforcement

(1) The inside radius of bends in reinforcement along the outer side of a corner in a frame structure shall be not less than 10 times the reinforcement diameter.

(2) Reinforcement along the inner sides of corners in a haunch or rigid frame shall not be bent reinforcement carrying tension of slabs or beams.

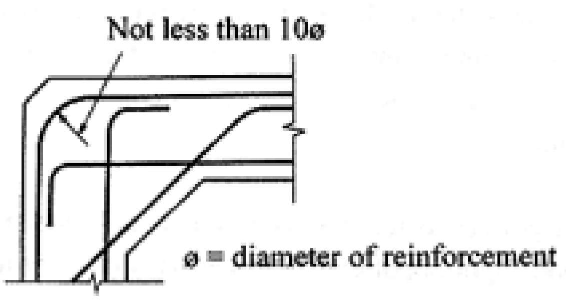


Fig. 10.4.1: Inside radius of bend in reinforcement along outer side of corner in frame structure

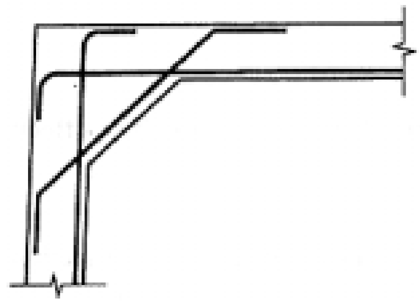


Fig. 10.4.2: Reinforcement along inner side of corner in haunch or frame structure

10.5 ANCHORAGES

10.5.1 General

(1) Reinforcement ends shall be embedded sufficiently in concrete, and anchoring shall be achieved either by the bonding force between the reinforcement and concrete, or by mechanical anchoring.

(2) At least 1/3 of the positive moment reinforcement in slabs or beams shall be anchored beyond the support, without being bent.

(3) At least 1/3 of the negative moment reinforcement in slabs or beams shall extend beyond the inflection point and anchored in the compression zone, or shall be connected to the next negative moment reinforcement.

(4) Stirrups shall enclose positive or negative moment reinforcement, and their ends shall be either closed or anchored in the concrete on the compression side.

(5) Spiral reinforcement shall be anchored in concrete enclosed by spiral reinforcement wound an extra one and a half turns.

(6) When the end of reinforcement is anchored by bonding between concrete and reinforcement, anchoring shall be done following the development length given in **10.5.2**.

[COMMENTS]:
In CFRM reinforced concrete, the CFRM and concrete must act in concord against external forces. Thus, when there is an external force acting against concrete members, the anchoring of the reinforcement is extremely important, and must be developed free from defects. If the anchoring of the reinforcement ends is adequate, the effects of local bond may be ignored, thus in this section only development of bar ends is covered.

(1) CFRM may be categorized as follows according to their bond property.
[1] Bond failure by bond splitting of concrete: This is equivalent to the failure mode of deformed steel bars, and in general, this is the mode of failure observed when the surface of the CFRM is treated to resemble a deformed steel bar.

[2] Bond failure by pull-out of reinforcement: This mode of failure is generally observed where indentations on the surface of the CFRM are small, or where abrasive grains or threads are bonded onto the CFRM surface, but the bond strength is low.
[3] No bond strength: CFRM with smooth surfaces generally has lower bonding action with concrete than conventional round steel bars, giving almost no bond strength at all. In these cases mechanical anchoring is required.
[4] Anchoring by resistance from intersecting lateral reinforcement: In lattice and solid configurations, anchoring is generally achieved by the resistance of intersecting lateral reinforcement.

In order to achieve full strength of reinforcement, depending on the bond characteristics of the CFRM used, either an adequate development length should be allowed or a mechanical anchorage fitted to embed the reinforcement securely within the concrete, in order to ensure the CFRM does not pull out from the concrete. Given that CFRM looses strength in bent sections, and that their flexural rigidity is inadequate, unlike the case with steel reinforcement no anchoring effect is expected from hooks.

Where bond between the reinforcement and the concrete is relied on for anchoring, reinforcement must also be arranged perpendicularly, to ensure adequate anchoring. For tensile reinforcement at the fixed ends of members, both ends of tensile reinforcement in footings, tensile reinforcement at the free ends of cantilever beams and so forth, anchorages should be fitted to prevent reinforcement pulling out even if major cracking appears.

(4) When a diagonal crack occurs in a beam, the two parts of the beam on either side of the crack will tend to part from one another. Stirrups are place to prevent these two parts from separating, performing the function of a vertical tensile member of a Howe truss. The stirrup must therefore either be closed, or bent so that its end is hooked around reinforcement in the compression zone, to ensure that its end is properly anchored. The purpose of enclosing compression reinforcement with stirrups is to anchor the stirrup properly, and to prevent the compression reinforcement from buckling.

10.5.2 Development length of reinforcement

(1) The development length for CFRM l_0 shall be not less than the basic development length l_d. Where the quantity of reinforcement placed A_f is greater than the quantity required by calculation A_{fc}, development length l_0 may be reduced in accordance with Eq. (10.5.1)

$$l_0 \geq l_d \cdot (A_{fc} / A_f) \qquad (10.5.1)$$

where

$l_0 \geq l_d/3$, $l_0 \geq 10ø$
ø = diameter of reinforcement

(2) The development length of reinforcement where the anchorage is bent shall be as follows:
(i) When the inside radius of the bend is not less than 10 times the reinforcement diameter, the entire length of reinforcement including the bent part shall be effective.
(ii) When the inside radius of the bend is less than 10 times the reinforcement diameter and the straight part beyond the bend is extended more than 10 times the reinforcement diameter, only the straight part beyond the bend shall be effective.

(iii) The length of the straight part l' shall be not less than the length necessary for the stress acting on the reinforcement in the bent part not to exceed the tensile strength of the bent part.

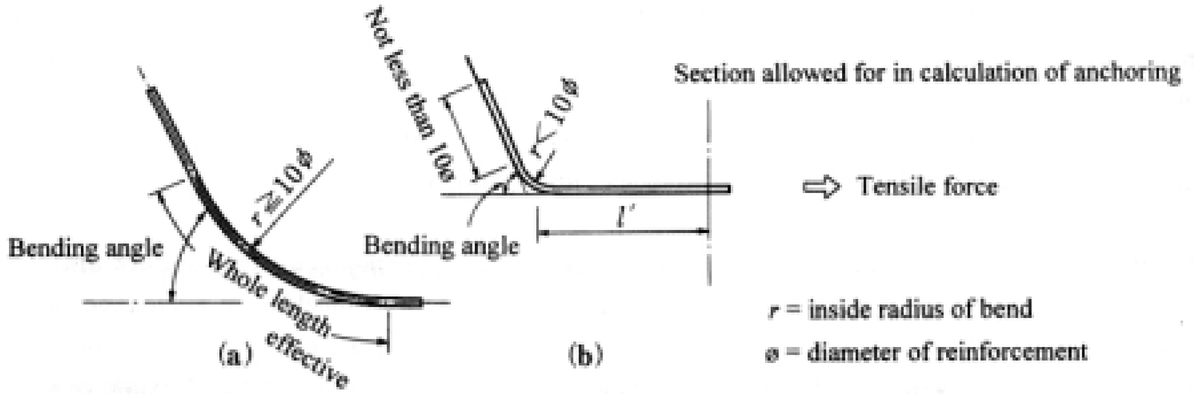


Fig. 10.5.1: Determination of development length of reinforcement in bent anchorages

(3) Tensile reinforcement shall generally be anchored in concrete not subject to tensile stress. If either of the conditions (i) or (ii) below is satisfied, tensile reinforcement may be anchored in concrete subject to tensile stress. In this case, the anchorage of the tensile reinforcement shall be extended by (l_d + l_s) from the section where the reinforcement is no longer required to resist calculated flexure, where l_d is the basic development length and l_s may in general be the effective depth of the member section.

(i) The design shear strength shall be not less than 1.5 times the design shear force between the point of reinforcement cutoff and the section where the reinforcement is no longer required to resist calculated flexure.

(ii) The design flexural capacity shall be not less than 2 times the design moment at a point where adjacent reinforcement terminates, and design shear capacity shall be not less that 4/3 times the design shear force between the point of reinforcement cutoff and the section where the reinforcement is no longer required to resist calculated flexure.

(4) Where positive moment reinforcement in a slab or beam is anchored beyond the support at the end, the development length of the reinforcement shall be not less than l_0 for stress in reinforcement at a section which is at a distance of l_s from the center of the support and shall be extended to the end of the member.

[COMMENTS]:

(1) The development length is calculated from the basic development length l_d, determined by the type and arrangement of the reinforcement, and by the strength of the concrete, modified according to the usage conditions.

Where the quantity of reinforcement placed is in excess of that quantity required according to calculation, the basic development length may be reduced proportionally. A minimum value for l_0 has been given, as the safety level with regard to additional forces is reduced.

(2) (iii) As the tensile strength of bent sections of CFRM is generally less than that of straight sections, it is necessary to reduce the tensile force acting on the bent section by the bonding at the straight length l'. Where the quantity of reinforcement placed is in excess of that quantity required according to calculation, length l' may be reduced following section (1) above.

10.5.3 Basic development length

(1) The basic development length of CFRM shall generally be determined on the basis of appropriate testing.

(2) The basic development length of tensile reinforcement types which undergo bond splitting failure may be calculated according following Eq. (10.5.2), provided that ld > 20ø.

$$l_d = \alpha_1 \frac{f_d}{4 f_{bod}} \phi \qquad (10.5.2)$$

where

ø = diameter of main reinforcement

f_d = design tensile strength of CFRM

f_{bod} = design bond strength of concrete according to Eq. (10.5.3), where $\gamma_c = 1.3$

$$f_{bod} = 0.28\alpha_2 f'_{ck}{}^{2/3} / \gamma_c \ \ (\text{N/mm}^2) \qquad (10.5.3)$$

where

$f_{bod} \leq 3.2$ N/mm^2

α_2 = modification factor for bond strength of CFRM; $\alpha_2 = 1.0$ where bond strength is equal to or greater than that of deformed steel bars; otherwise α_2 shall be reduced according to test results.

f'_{ck} = characteristic compressive strength of concrete

$\alpha_1 = 1.0$ (where $k_c \leq 1.0$)
$= 0.9$ (where $1.0 < k_c \leq 1.5$)
$= 0.8$ (where $1.5 < k_c \leq 2.0$)
$= 0.7$ (where $2.0 < k_c \leq 2.5$)
$= 0.6$ (where $2.5 < k_c$)

where

$$k_c = \frac{c}{\phi} + \frac{15A_t}{s\phi} \cdot \frac{E_t}{E_0} \qquad (10.5.4)$$

c = downward cover of main reinforcement or half of the space between the anchored reinforcement, whichever is the smaller

A_t = area of transverse reinforcement which is vertically arranged to the assumed splitting failure surface

s = distance between the centers of the transverse reinforcement

E_t = Young's modulus of transverse reinforcement

E_0 = standard Young's modulus (= 200 kN/mm^2)

(3) Where the reinforcement to be anchored is located at a height of more than 30 cm from the final concrete surface during concrete placement and at an angle of less than 45° from the horizontal, the basic development length shall be 1.3 times the value of l_d obtained from the application of section (2).

(4) The basic development length of compression reinforcement shall be 0.8 times the values of l_d obtained from the application of sections (1), (2) and (3).

[COMMENTS]:

(1) The development length of CFRM varies with the reinforcement type, concrete strength, concrete cover and transverse reinforcement. These factors must be adequately allowed for in testing. For this reason, the test method(s) used to determine the development length of a CFRM should be methods which reflect the actual bond characteristics within the member, such as methods using test beams or lap jointed test specimens.

JSCE-E 539 "Test Method for Bond Strength of Continuous Fiber Reinforcing Materials by Pull-Out Testing" does not reflect the actual bond characteristics within the member, and thus will generally over-estimate bond strength. Calculation of basic development length substituting bond strengths obtained from this test for f_{bod} should thus be avoided.

(2) In the JSCE Standard Specification (Design), the required development length for steel reinforcement with transverse reinforcement is given as Eq. (C 10.5.1)

$$l_0 = \frac{\left(\dfrac{f_{yd}}{1.25\sqrt{f'_{cd}}} - 13.3\right)\phi}{0.318 + 0.795\left(\dfrac{c}{\phi} + \dfrac{15A_t}{s\phi}\right)} \qquad \text{(C 10.5.1)}$$

where

f_{yd} = design tensile yield strength of steel reinforcement (N/mm^2)

f'_{cd} = design compressive strength of concrete (N/mm^2)

$c/\phi \leq 2.5$

This equation is further simplified by factoring in a factor α, given in the present recommendation.

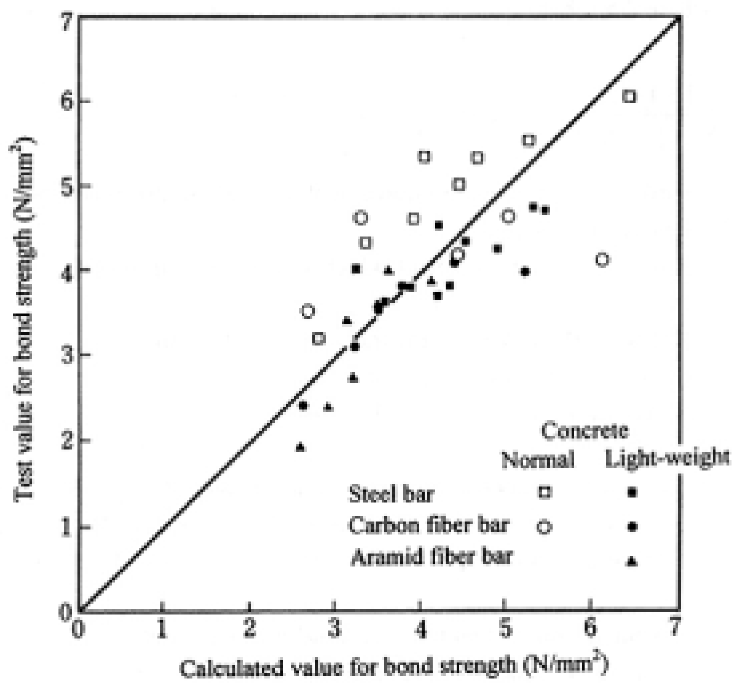


Fig. C 10.5.2 Comparison of bond strength Eq. (C 10.5.2) with test results

For CFRM with deformed surfaces which fail by bond splitting, comparison of the bond strength obtained from testing of this bond splitting and the bond strength calculated according to the formula below, derived allowing for the ratio of the Young's modulus of the CFRM used as transverse reinforcement E_t (=E_f) to the standard Young's modulus E_0 (=E_s) yields the following formula:

$$f_{bod} = \frac{0.318 + 0.795\left(\dfrac{c}{\phi} + \dfrac{15A_t}{s\phi}\cdot\dfrac{E_t}{E_0}\right)}{\dfrac{3.2}{\sqrt{f'_c}} - \dfrac{53.2}{f_y}} \qquad \text{(C 10.5.2)}$$

Based on Eq. (**C 10.5.2**), evaluation of the basic development length according to the method used for deformed steel bar has been allowed for any CFRM that fails by bond splitting. For CFRM that fail by bond splitting but show bond strength that is not equal to or greater than that of deformed steel bars, if the design bond strength is estimated following Eq. (10.5.3), a modification factor α_2 ($\leq$1.0) shall be factored in. Where the data available is inadequate or where significant variation is found, the basic development length shall generally be determined by appropriate testing.

The basic development length of reinforcement where the bond failure mode is by pull-out may be determined by appropriate testing.

10.6 SPLICES

10.6.1 General

It shall be in accordance with JSCE Standard Specification (Design), 9.6.1, where "steel" shall be taken to signify "steel or CFRM".

10.6.2 Lap splices

Lap splices for longitudinal reinforcement shall follow the JSCE Standard Specification (Design), 9.6.2, where "steel" shall be taken to signify "steel or CFRM".

[COMMENT]: Where the quantity of reinforcement place A_f is greater than the calculated requirement A_{fc}, the length of lap splices may be reduced by factoring in A_{fc}/A_f, determined as specified in the present recommendation (above). Where $A_{fc}/A_f > 350/f_d$, $A_{fc}/A_f = 350/f_d$, where f_d = design tensile strength of CFRM (N/mm^2).

CHAPTER 11: PRESTRESSED CONCRETE

11.1 GENERAL

(1) This chapter gives general guidelines required for the design of prestressed concrete structures or members with CFRM tendons or CFRM tendons in conjunction with steel tendons.

(2) Prestress levels shall be determined to ensure that the structure or member can fulfill its purpose safely and economically and give the desired performance.

[COMMENTS]:

Prestressed concrete structures make possible improvement of the crack characteristics at the serviceability limit state as well as reduction of the cross sectional area of a member, thus offering an extended range of options for many types of structure.

Design calculations of prestress force in concrete members are generally handled by regarding prestress forces in the serviceability limit state as loads, considering only the statically indeterminate forces in loads when the effects of prestress forces are included in calculation of cross-sectional bearing capacity at ultimate limit state. In this case, the ultimate strain of CFRM has to be checked.

Where member forces are calculated other than by linear analysis, prestress forces must be handled in the appropriate manner for the analysis method used.

(1) The provisions of this chapter shall be applied to general prestressed concrete structures or members with CFRM tendons or CFRM tendons in conjunction with steel tendons. These provisions shall not be applied to the following types of structures or members:

[1] Structures or members where the prestress force is transferred by a method other than prestressing tendons; for example methods to use a jack to give prestress to arch members or concrete pavement, methods using expanding agent to introduce prestressing, or methods whereby concrete is cast on the tension side of a steel girder to which flexural moment has previously been applied, subsequently releasing the moment to induce prestressing after the concrete has hardened.

[2] Prestressed steel reinforced concrete structures or members; prestressed composite structures or members consisting of steel and concrete.

[3] Prestressed concrete members or structures constantly exposed to abnormal temperatures, where "normal temperature" is taken to signify a temperature within the range 0°C~40°C.

[4] Factory products such as prestressed concrete piles, prestressed concrete pipes, etc.

For structures or members having unbonded prestressing tendons ("unbonded CFRM") and structures or members wherein which CFRM is used as external cables, considerations other than those given here must be allowed for; these will include the increase in flexural cracking widths due to the lack of bond with the concrete members, the reduction of flexural capacity, the minimum reinforcement quantity, fatigue resistance of the anchoring devices etc. In structures with external cables which are exposed to a fire risk, fireproofing measures will be required.

(2) Prestressing may generally be determined for limit states at the tensile edge, according to the function of the structure (cf. 11.2)

11.2 CATEGORIZATION OF PRESTRESSED CONCRETE

It shall be in accordance with JSCE Standard Specification (Design), 10.2.

11.3 PRESTRESS FORCE

(1) Prestress force shall be calculated according to Eq. (11.3.1).

$$P(x) = P_i - \left[\Delta P_i(x) + \Delta P_t(x)\right] + \Delta P_T(x) \qquad (11.3.1)$$

where

$P(x)$ = prestress force in design section under consideration

P_i = prestress force during prestressing due to tension applied to tendon ends

$\Delta P_i(x)$ = loss of prestressing force during and immediately after prestressing, to be calculated allowing for the following effects:

[1] elastic deformation of concrete
[2] friction between tendon and duct
[3] set loss on anchoring of tendon
[4] other considerations

$\Delta P_t(x)$ = time-dependent loss of prestressing force, to be calculated allowing for the following effects:

[5] relaxation of tendon
[6] creep of concrete
[7] shrinkage of concrete

$\Delta P_T(x)$ = variation of prestress force due to temperature change

(2) In calculating the indeterminate forces at the serviceability limit state or the fatigue limit state, the prestress force given by Eq. (11.2.1) may be considered to be the characteristic value for the prestress forces.

[COMMENTS]:

(1) The effects to be considered in calculating prestress losses $\Delta P_i(x)$ and $\Delta P_t(x)$ in relation to calculation of prestress force.

[1] Effects of elastic deformation of concrete: Prestress loss due to elastic deformation of concrete shall always be considered when the pretensioning system is used (c.f. Eq. (C 11.3.1)). When post-tensioning tendons are tensioned consecutively, the prestress loss due to elastic deformation of concrete shall be calculated. In such cases, the average prestress loss may be calculated in accordance with Eq. (C 11.3.2).

(pretensioning) $\Delta\sigma_p = n\sigma'_{cpg}$ (C 11.3.1) (post-tensioning) $\Delta\sigma_p = \frac{1}{2} n \sigma'_{cpg} \frac{N-1}{N}$ (C 11.3.2)

where

$\Delta\sigma_p$ = prestress loss in tendon
n = Young's modulus ratio E_p/E_c
σ'_{cpg} = compressive stress of concrete at tendon centroid due to tensioning
N = number of tendon tensionings (i.e. number of tendon groups)
E_c = Young's modulus of concrete
E_p = Young's modulus of tendons; for CFRM $E_p = E_f$

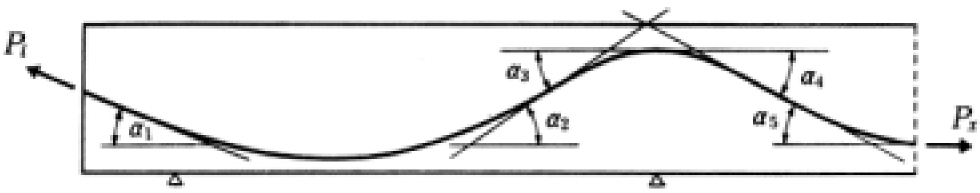
[2] Effects of friction between tendon and duct: prestress loss in prestressing tendons due to friction depends on the condition of the inner surface of the duct, the type of prestressing tendon, and the positioning of tendons.

Loss of prestressing tendon force due to friction can generally be divided into two terms, one related to the angular change of the centroid line of the prestressing tendons, and the other related to the length of the prestressing tendons. The tensile force in the prestressing tendon at the design section may be expressed by Eq. (C 11.3.3).

$P_x = P_i e^{-(\mu\alpha+\lambda x)}$ (C 11.3.3)

where

P_x = tensile force of tendon at design section
P_i = tensile force at position of prestressing jack
μ = coefficient of friction per radian of angular change
α = angular change (radians) (c.f. **Fig. C 11.3.1**)
λ = coefficient of friction per unit length of tendon
x = distance from tensioned edge of tendon to design section



$\alpha = \sum_1^n \alpha_i = \alpha_1 + \alpha_2 + \alpha_3 + \alpha_4 + \alpha_5 + \cdots\cdots\cdots \alpha_n$

Fig. C 11.3.1: Angular change of tendon centroid line

The values of μ and λ will vary depending on the tendon and sheath material, and must therefore be determined by testing, but where sheaths are used with CFRM, the tensile force in the tendon may generally be calculated with $\mu = 0.3$, $\lambda = 0.004$.

[3] Effects of set loss on anchoring of tendon: Where there is set loss during anchoring of tendons, the resultant prestress loss must be allowed for. Set loss is especially significant with wedge-type anchorage systems, therefore the prestress loss and the length affected by it must be examined prior to tensioning by applying the assumed set loss based on available data. "Set loss" refers to the pulling of a tendon into

the anchoring device during anchoring. The amount of set loss varies depending on the type of anchoring device, and must therefore be studied separately for each type.

Where there is no friction between the prestressing tendons and the duct, loss of prestressing force due to set loss is calculated according to Eq. (C 11.3.4).

$$\Delta P = \frac{\Delta l}{l} \times A_p E_p \qquad \text{(C 11.3.4)}$$

where

ΔP = loss of prestressing force due to set loss
Δl = set loss
l = length of tendon
A_p = cross-sectional area of tendon

Where there is friction between the prestressing tendons and the duct, loss of prestressing force in the tendons may be calculated as follows. Assuming identical frictional force during tensioning and releasing, the distribution of tendon force is as shown in **Fig. C 11.3.2**. When a tendon is tensioned from end a, the prestressing force in the tendon is a'b'co' immediately prior to anchoring, and the prestressing force at the tensioning end immediately after to anchoring decreases to P_t. In this case, lines a'b'c and a"b"c are symmetrical with respect to the horizontal axis ce, and the amount of set is equal to the area A_{ep} enclosed by a'b'cb"a", divided by A_pE_p.

$$\Delta l = \frac{A_{ep}}{A_p E_p} \qquad \text{(C 11.3.5)}$$

Thus, the line cb"a" may be obtained by determining the point c where A_{ep} is equal to $\Delta l A_p E_p$.

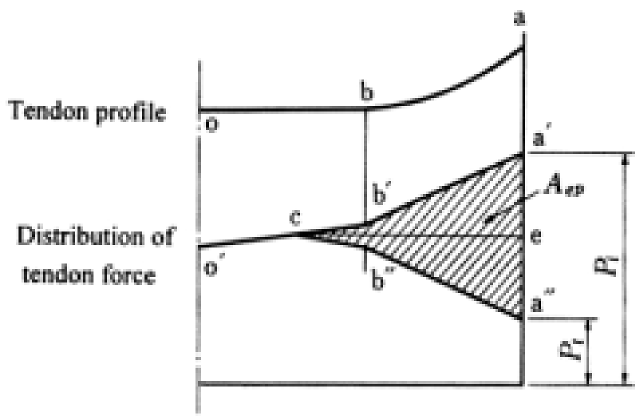


Fig. C 11.3.2: Distribution of tendon force

[4] Other effects: these will include e.g. deformation of joints used in precast block construction

[5] Effects of relaxation of tendon: loss of prestressing force in tendons due to tendon relaxation may be obtained from Eq. (C 11.3.6).

$$\Delta\sigma_{pr} = \gamma\sigma_{pt} \qquad \text{(C 11.3.6)}$$

where

$\Delta\sigma_{pr}$ = loss of prestressing force in tendons due to tendon relaxation

γ = apparent relaxation rate in tendon

[6] Creep of concrete, [7] Shrinkage of concrete: loss of prestressing force in tendons due to creep and shrinkage of concrete are determined on the basis of appropriate creep analysis; in general **COMMENTS** (1) (ii) to section **11.4** below may be applied.

Owing to the difference between the thermal expansion coefficients and the Young's modulus of concrete and CFRM, the prestressing of CFRM varies with temperature; for example a prestress loss of around 2% is found for a temperature rise of 20°C in the case of carbon fiber CFRM. This effect must therefore be allowed for where major temperature variations are expected. Prestress loss due to temperature change may be obtained from Eq. (C 11.3.7).

$$\Delta\sigma_{pT} = \Delta T(\alpha_f - \alpha_{CON})E_f \qquad \text{(C 11.3.7)}$$

where

$\Delta\sigma_{pt}$ = prestressing force loss in CFRM due to temperature change

ΔT = temperature change

α_{CON} = thermal expansion coefficient of concrete

α_f = thermal expansion coefficient of CFRM

E_f = Young's modulus of CFRM

(2) For indeterminate structures, it is possible to prevent indeterminate forces due to prestressing by selecting the appropriate tendon arrangement. Generally, though, indeterminate forces occur when member deflection due to prestress force is restricted, and these indeterminate forces must be allowed for when calculating stresses acting on the cross sections.

It should be noted that the level of indeterminate forces due to prestress forces are significantly affected by changes in the cross-sectional area of the member.

11.4 SERVICEABILITY LIMIT STATE

11.4.1 Flexural moments and axial forces

(1) Stress calculation

(i) Stress calculation

Stress in concrete, CFRM and steel shall be calculated according to **7.2**, based on the following assumptions etc.:

[1] In prestressed concrete structures, the entire concrete section is effective

[2] In prestressed reinforced concrete structures, the tensile stress of concrete shall generally be ignored

[3] Strain increase in bonding tendons is identical to that at the same position in concrete

[4] Axial ducts in members are not considered part of the effective cross-section

[5] The section constant of the integrated tendons and concrete shall be determined allowing for the

Young’s modulus ratio of the tendons and the concrete.

(ii) Stresses in concrete, CFRM and steel subjected to permanent load shall be determined allowing for the effects of tendon relaxation, creep and shrinkage of concrete, and the constraining effect of steel.

(iii) Stresses in concrete, CFRM and steel subjected to variable loads shall be determined based on the stress under permanent loads calculated in (ii) above.

(2) Limiting values of stress

Limiting values for compressive stress in concrete due to flexural moment and axial forces shall be according to section **7.3** above. Limiting values for tensile stress in tendons shall be determined based on testing, according to the type of tendon sued. Limiting values for prestressing steel shall be according to JSCE Standard Specification(Design), 10.4.

(3) Examination for prestressed concrete structures

(i) Limiting values for edge tensile stress in concrete shall be the design tensile strength, allowing for the effects of member dimensions.

(ii) Where edge tensile stress of concrete acts as tensile stress, in general a quantity of tensile steel in excess of the cross-sectional area calculated according to Eq. (11.4.1) shall be arranged. Deformed bars shall generally be used for steel reinforcement.

$$A_s = T_c / \sigma_{st} \qquad (11.4.1)$$

where

A_s = cross-sectional area of tensile steel

T_c = total tensile force acting on concrete

σ_{st} = limiting value for tensile stress of tensile steel, may be set at 200 N/mm^2 for deformed bars. Bonding prestressing steel arranged in concrete where tensile stresses occur may be regarded as tensile steel. In this case, the limiting value for tensile stress of prestressing steel used in pretensioning may be set at 200 N/mm^2, and at 100 N/mm^2 for prestressing steel used in post-tensioning.

(4) Examination for prestressed reinforced concrete structures
Verification of flexural cracking shall be according to section **7.4** above.

(5) Verification of deflection shall be according to section **7.5** above, allowing for the effects of prestressing.

[COMMENTS]:
(1)(i): [3] In calculating strain increase in external cable tendons and unbonded tendons (i.e. unbonded CFRM or unbonded prestressing steel), the assumption of “plane remains plane” cannot be applied, therefore separate study is required. In this case, concrete stress may be calculated ignoring stress increase in tendons, regarding the structure as a reinforced concrete structure subject to eccentric axial force due to the effective prestress force.

[4] Grouting in ducts is not subject to prestress, therefore ducts in the axial direction of the member should not be included in the effective section, even when grouted.

(1) (ii) Where there is bond between the concrete and the tendons, stresses in the concrete, tendons and steel under permanent load shall be calculated as follows.

[1] Prestressed concrete structures:
The constraining effect of steel may be ignored in prestressed concrete structures. The reduction in tensile stress of tendons may in this case be calculated according to Eq. (C 11.4.1):

$$\Delta\sigma_{pcs} = \frac{n_p \cdot \varphi(\sigma'_{cpt} + \sigma'_{cdp}) + E_p \cdot \varepsilon'_{cs}}{1 + n_p \cdot \dfrac{\sigma'_{cpt}}{\sigma_{pt}} \cdot \left(1 + \dfrac{\varphi}{2}\right)} \qquad \text{(C 11.4.1)}$$

where

$\Delta\sigma_{pcs}$ = tensile stress reduction in tendons due to concrete creep and shrinkage

φ = creep factor

ε'_{cs} = shrinkage strain of concrete

n_p = ratio of Young's modulus of tendons to that of concrete

σ_{pt} = tensile stress of tendons immediately after tensioning

σ'_{cpt} = compressive stress of concrete at tendons due to prestressing immediately after tensioning

σ'_{cdp} = compressive stress of concrete at tendons due to permanent load

[2] Prestressed reinforced concrete structures
In prestressed reinforced concrete structures, the constraining effect of steel is generally considered in calculations. In this case, the reduction of tensile stress in tendons and the stress change in tensile reinforcement may be calculated according to Eqq. (C 11.4.2) and (C 11.4.3). Stress in concrete where cracking does not occur under permanent loads shall be calculated allowing for the effects of the reaction force of the compressive forces acting on tensile reinforcement.

$$\{1 + \alpha_{pp} \cdot (1 + \varphi/2)\} \cdot \Delta\sigma_{pcs} + \alpha_{sp} \cdot (1 + \varphi/2) \cdot \Delta\sigma_{scs}$$

$$= n_p \cdot \{\varphi \cdot (\sigma'_{cpt} + \sigma'_{cdp}) + E_c \cdot \varepsilon'_{cs}\} \qquad \text{(C 11.4.2)}$$

$$\alpha_{ps} \cdot (1 + \varphi/2) \cdot \Delta\sigma_{pcs} + \{1 + \alpha_{ss} \cdot (1 + \varphi/2)\} \cdot \Delta\sigma_{scs}$$

$$= n_s \cdot \{\varphi \cdot (\sigma'_{cps} + \sigma'_{cds}) + E_c \cdot \varepsilon'_{cs}\} \qquad \text{(C 11.4.3)}$$

given that

$$\alpha_{pp} = n_p \cdot A_p \cdot (1/A_c + e_p^{\,2}/I_c)$$

$$\alpha_{ps} = n_s \cdot A_p \cdot (1/A_c + e_p \cdot e_s/I_c)$$

$$\alpha_{sp} = n_p \cdot A_s \cdot (1/A_c + e_p \cdot e_s/I_c)$$

$$\alpha_{ss} = n_s \cdot A_s \cdot (1 / A_c + e_s{}^2 / I_c)$$

where

$\Delta\sigma_{pcs}$ = tensile stress reduction in tendons due to concrete creep and shrinkage

$\Delta\sigma_{scs}$ = variation in tensile reinforcement stress due to concrete creep and shrinkage

φ = creep factor

ε'_{cs} = shrinkage strain of concrete

n_p, n_s = ratio of Young's modulus of tendons and steel to that of concrete

$n_p = E_p/E_c$; for CFRM $n_p = E_f/E_c$, $n_s = E_s/E_c$

σ'_{cpt} = compressive stress of concrete at tendons due to prestressing immediately after tensioning

σ'_{cps} = compressive stress of concrete at steel reinforcement due to prestressing immediately after tensioning

σ'_{cdp} = compressive stress of concrete at tendons due to permanent load

σ'_{cds} = compressive stress of concrete at steel reinforcement due to permanent load

A_p = cross-sectional area of tendons

A_s = cross-sectional area of steel reinforcement

e_p = distance from centroid axis of member section to centroid of tendon

e_s = distance from centroid axis of member section to centroid of steel reinforcement

A_c = total cross-sectional area of concrete

I_c = moment of inertia of total concrete section

The effective prestress of unbonded tendons and tendons used in external cabling can in theory be calculated by first determining the stress changes along the entire tendon length at the centroid of the tendon due to deformation of concrete members, then calculating stress changes in the tendon from the average strain. As this calculation would be enormously complex, while member deformation at the serviceability limit state is minimal and the effects of strain changes at the tendon positions are slight, Eq. (C 11.4.1) may also be applied to external cabling.

(2) If concrete cracking, prestressing steel fatigue etc. are studied, there is no particular need to limit tensile stress in concrete and prestressing steel, but once tensile stresses exceed the elastic limit, the assumptions made in structural analysis and stress calculation fail to hold good, and prestress force can no longer be treated as an external force. For this reason, tensile stress must be kept below the elastic limit stress. Consistency has also been allowed for in long-term relaxation testing of prestressing steel, where the initial load was set at 70% of the characteristic value of the tensile strength of prestressing steel.

Unlike reinforcing or prestressing steel, CFRM is liable to fail below their static strength (creep failure) when subjected for long periods to significant, sustained stress for long periods. When using CFRM tendons, therefore, the tension must be set allowing for the creep failure strength. Creep failure strength is calculated on the basis of JSCE-E 533 "Testing Method for Creep Failure of Continuous Fiber Reinforcing Materials", testing up to 1000 hours and extrapolating creep failure strength at 1 million hours. The limit value for tensile stress of tendons may generally be taken as the creep failure strength characteristic value f_{fck}, multiplied by a reduction factor of 0.8. The limit value shall be not more than 70% of tensile strength (c.f. 7.3).

(3)

(i) Limiting values for edge tensile stress in concrete for prestressed concrete structures is taken as being the design tensile strength, allowing for the effects of member dimensions according to Eq. (C 7.4.1) in the JSCE Standard Specification (Design).

Table C 11.4.1 gives limiting values for edge tensile stress.

Table C 11.4.1: Limiting values for edge tensile stress in concrete for prestressed concrete structures

Loading status	Section depth (m)	Design strength f'_{ck} (N/mm^2)					
		30	40	50	60	70	80
Subject to variable load	0.25	2.1	2.6	3.0	3.4	3.8	4.1
	0.5	1.7	2.1	2.4	2.7	3.0	3.3
	1.0	1.3	1.6	1.9	2.2	2.4	2.6
	2.0	1.1	1.3	1.5	1.7	1.9	2.0
	3.0 ~	0.9	1.1	1.3	1.5	1.7	1.8

(ii) Provision is made for calculation of the tensile steel quantity to be placed where the concrete is subject to tensile stress, minimizing the difference between section stress calculated assuming the entire concrete section to be effective and section stress calculated assuming cracking to be present.

The quantity of tensile steel may be calculated either by the method given here, or by a method ignoring the tensile stress of concrete. For tensile stresses up to the level of the design strength, the method given here has been adopted as it is simpler and also more conservative.

If CFRM tendons are considered as being equivalent to tensile reinforcement, the strain limit value may be substituted for the limiting value for stress in prestressing steel, and this strain limit value applied to CFRM.

A distinction is drawn between pre- and post-tensioning tendons, to allow for the different bond strengths to the concrete in the two systems.

When unbonded tendons are used, the required quantity of tensile steel may be calculated multiplying the value by 1.35 with regard to variable loads. Unbonded tendons are not regarded as tensile reinforcement, as in prestressed concrete with small quantities of steel reinforcement may be liable to significant crack concentrations with attendant steel corrosion.

Concrete members used in external cabling systems are to be arranged with the minimum quantity of steel for reinforced concrete members subject to flexure. No additional variable loads are to be factored into the calculation of tensile steel quantities. Tendons placed as external cables are not regarded as tensile reinforcement.

(4) For prestressed reinforced concrete structures, a limit value for crack width shall be determined based on environmental conditions, the function and purpose of the structure or member, etc., and flexural cracking studied according to **7.4**. In this case, the increase in tensile reinforcement stress may be calculated according to (1)(iii), and flexural crack widths calculated with regard to durability according to Eq. (7.4.1).

In prestressed reinforced concrete structures, unbonded tendons, external cabling tendons etc. are used in conjunction with deformed steel bars. Corrosion-proofing for prestressing steel is considered separately, while CFRM is not liable to corrosion. As flexural crack widths are normally studied with regard to the outermost layer of deformed steel bars, the standard method of calculation of flexural crack widths may be adhered to.

11.4.2 Shear and torsion

It shall be in accordance with JSCE Standard Specification (Design), 10.4.2.

11.5 ULTIMATE LIMIT STATE

Verification of ultimate limit state for sectional failure shall be done in accordance with Chapter 6.

[COMMENTS]:
When unbonded tendons are used, flexural capacity is generally lower than when bonding tendons are used. For this reason, flexural capacity shall be reduced by 30% except in special cases where allowance is made for tensile stress of tendons, tendon layout, flexural moment distribution, coefficient of friction between tendons and concrete etc.

When ducts etc. are provided perpendicular to the member axis for transverse reinforcement etc., the duct sections need not be deducted from the concrete section. Duct sections shall be deducted from the concrete section in the following cases:

[1] When ducts etc. are not grouted
[2] When the diameter of the ducts etc. exceeds 30% of the member thickness

11.6 FATIGUE LIMIT STATE

It shall be in accordance with JSCE Standard Specification (Design), 10.6, where "steel" shall be taken to signify "steel or CFRM".

11.7 SAFETY DURING CONSTRUCTION

The following studies shall generally be made in relation to construction:

(1) Tensile stress in tendons during or immediately following prestressing shall be determined by testing, allowing for variations in material strengths.

(2) In verifying the safety of flexural moments and axial loads, the tensile stress of the concrete shall be not greater than the design tensile strength of the concrete, allowing for scale effect in the members. The design tensile strength of the concrete may be determined using the characteristic value for the compressive strength of the concrete at the time of calculation, taking g_c as equal to 1.0.

In concrete regions subject to tensile stress, tensile reinforcement having a cross-sectional area of not less than 3/4 of that calculated according to **11.4.1(3)** shall be arranged.

(3) Flexural compressive stress and axial compressive stress in concrete due to flexural moment and axial forces immediately following prestressing shall be respectively not more than 1/1.7 and 1/2 of the characteristic values for compressive strength of the concrete.

(4) Deflection shall be verified in accordance with Section **7.5** above, allowing for the effects of prestressing.

(5) The effects of shear and torsion shall be verified in accordance with Section **11.4.2** above. The design tensile strength of the concrete may be determined using the characteristic value for the compressive strength of the concrete, taking γ_c as equal to 1.0.

(6) Verification of the ultimate limit state may be done if necessary in accordance with Chapter 6.

[COMMENTS]:
The expression "during construction" shall generally be taken to signify the period during prestressing, immediately after prestressing, and subsequent stages until the structure goes into service.

(1) The limiting values for tensile stresses during and immediately after prestressing (respectively prestress forces P_i and (P_i-$\Delta P_i(x)$ in Eq. (11.3.1)) are to be determined by testing, given that they vary for different materials, and that tensile strength is subject to significant fluctuations. When carbon fiber CFRM is used, these values will generally be 0.7 f_{puk} and 0.65 f_{puk} respectively.

(2) Cracking during construction is generally not permitted, for the following reasons:
[1] Difficulty in controlling the width of cracks occurring during construction
[2] Difficulty in controlling deformation after loss of rigidity due to cracking
[3] Lack of data concerning shrinkage and creep behavior in the compressive zone of cracked concrete at the serviceability limit state

When all of the above issues have been adequately resolved, cracking during construction may be permitted. The limit value for flexural tensile stress has been determined based on considerations such as load combinations during construction, magnitude of flexural tensile stress, timing of the onset of flexural tensile stress etc. With regard to short-term tensile stresses, given that the section affected changes to compressive stress in service state, and also that the quantity of tensile steel in the service state is calculated separately, the quantity of tensile reinforcement may be reduced by 1/4, as provided for in this section. This reduction, however, is not recommended for tensile reinforcement subject to long-term tensile stresses, as the crack widths may grow due to creep of concrete.

11.8 STRUCTURAL DETAILS

11.8.1 Prestressing Tendons

(1) Clear distance
(i) The clear distance between sheaths for post-tensioning tendons shall satisfy the following

requirements:
[1] Horizontal and vertical spacing between sheaths shall be not less than 4/3 times the maximum size of the coarse aggregate;
[2] In areas where an internal vibrator is inserted, the horizontal spacing of sheaths or groups of sheaths shall be not less than 6cm, and the necessary spacing for the internal vibrator shall be maintained;
[3] Small size sheaths may be arranged in contact with each other if this is unavoidable. In such cases, the maximum number of sheaths shall be two, in the vertical direction;
[4] The vertical spacing between sheaths or groups of sheaths should be not less than the vertical section of the sheath

(ii) In pre-tensioning systems, the horizontal and vertical spacing of tendons at member ends shall be not less than 3 times the diameter of the tendon, and the horizontal spacing shall be not less than 4/3 times the maximum size of the coarse aggregate. When tendons are bundled in regions other than member ends, the numbers of layers ad tendons in a group shall be not more than 2 layers and 4 tendons respectively, and the spacing between each group shall be not less than 4/3 times the maximum size of the coarse aggregate.

(2) Concrete cover
Cover for a tendon, sheath or group of sheaths, and anchoring device shall be not less than the values given in Section **10.2**.

(3) Arrangement
(i) Tendons shall be so arranged that the prestress loss due to friction is low, and that there is no abrupt change in the cross-sectional area of the tendons throughout the member length.

(ii) Prestressing tendons shall be extended straight with the required length from the bearing face of the anchoring device.

(iii) When straight CFRM is arranged in a curve, the minimum curve radius to which the material may be bent by elastic deformation without causing damage shall first be determined by testing. Measures must be taken to eliminate any discrepancy between design and construction regarding the bending radius, allowing for the effects of secondary stresses on the strength of the reinforcement. The radius shall be determined such that excessive bearing pressure is not exerted on the concrete.

(iv) In the vicinity of section where moment reversal occurs due to the combination of loads, tendons should be dispersed between the upper and lower edges of the member section, avoiding concentrations of tendons near the centroid of the section.

(v) At an end support of a girder, some of the tendons should be extended along the lower face and anchored near the lower edge of the girder end.

(4) Arrangement of anchoring devices and couplers
(i) Anchoring devices shall be arranged so that each design section is subjected effectively to the necessary prestress, and that the tendons are securely fastened. Couplers shall be so arranged that the tendons can be securely coupled.
(ii) When multiple anchoring devices are arranged in the same section, the section configuration and

dimensions of the concrete in the anchorage region shall be determined allowing for the number of anchorage devices, the tendon forces and the required minimum spacing of the devices.

(5) Reinforcement for concrete adjacent to anchorage regions
Concrete adjacent to anchorage regions shall be reinforced with steel or CFRM, to prevent development of harmful cracks due to tensile stress.

(6) Protection of anchorages
Anchorages of tendons shall free from damage or corrosion for the duration of the design service life of the structure.

[COMMENTS]:

(1) (i) Spacing between sheaths shall generally be determined based on the following considerations.

Sheaths should be arranged as described in clauses [1] and [2], and group arrangements should if possible be avoided, in order to ensure that the concrete fully encloses the sheaths and that the entire cross section is filled with concrete.

Where group arrangement of sheaths is unavoidable due to limited member thickness and requirements for insertion of internal vibrators etc., vertical arrangements of up to two sheaths in the vertical direction as described in clause [3] may be used, provided that the sheaths are small and that special considerations are applied. A "small" sheath shall be one with a diameter of not more than approximately 70mm. "Special considerations" refers to sectional properties used in stress calculations, spacing of sheath bend-up locations, concrete casting method, concrete quality etc.

When a sheath is bent, the concrete between sheaths must be capable of withstanding the bearing stress of the tendons acting on the sheath walls. Where sheaths in the direction of the bearing stress at a bend are ungrouted as shown in **Fig. C 11.8.1(a)**, damage may occur if the clear distance is too small. Generally the clearance should be not less than one diameter of the sheath, as described in clause [4] (c.f. **Fig. C 11.8.1(b)**).

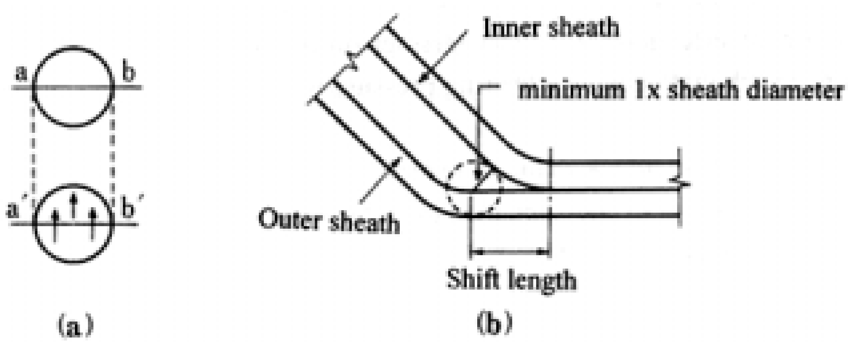


Fig. C 11.8.1: Arrangement of curved sheaths

(ii) In pre-tensioning systems, significant bond stress acts between the concrete and the tendons especially at the member ends. The clear spacing required here is given to ensure the development of adequate bond resistance, and adequate compaction of the concrete.

(3)
(i) Prestress loss due to friction is proportional to the angle change in the tendons and to tendon length, For bend-up or bend-down of tendons such as those in continuous girders, therefore, the effects of friction loss are considerable.

(iv) In continuous girders etc., in zones where loading causes reversal of bending moments, concentration of tendons around the center of the member section will reduce the quantity of reinforcement at the member edges, making cracking more likely. This should be prevented by distributing tendons in the zones close to the upper and lower edges (c.f. **Fig. C 11.8.2**).

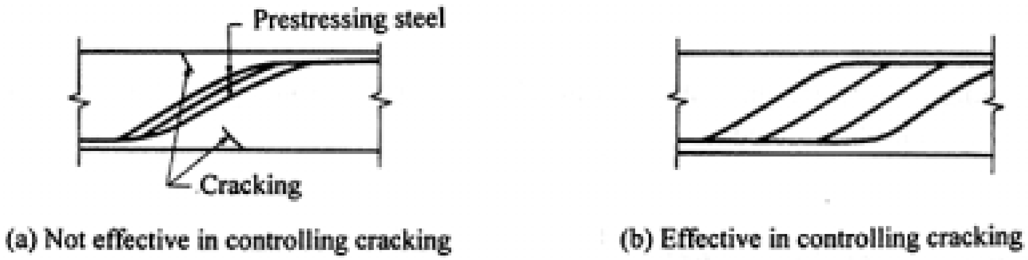


Fig. C 11.8.2: Arrangement of tendons in zones subject to moment reversal due to loading

(v) Where tendons cannot be arranged as described in this clause, the tendons must be replaced by axial reinforcement.

(4)
(i) In sections around anchorages, propagation of prestress and other effects cause stress disturbance, therefore the section cannot be treated as a normal section subject to eccentric axial loads for the purposes of stress calculation. When the design section is in the vicinity of an anchorage, the calculated prestress is not exerted, therefore the design section and the anchorage must be sufficiently separated to ensure prestressing is exerted effectively.

When an anchorage is placed in the central part of a member, it should generally be in the compression zone of the member.

The fatigue strength of an anchorage due to repeated loads is generally lower than the fatigue strength of tendons, therefore when an anchoring device is placed in the center of a member, it should located in a position where stress variation is at a minimum, and sufficiently removed from positions subject to large stress variation.

Couplers should be placed either in the vicinity of the centroid of the section, or in positions where bending moment variation is low.

For coupling in bent regions, tendons should be kept straight for a certain distance on either side of the coupler, and the coupling must be kept in a straight line.

(ii) The required minimum spacing of anchorages and the minimum concrete cover shall be determined by testing. Where the anchoring technique adopted is conventional and known to be sufficiently safe, the conventional practices may be adopted in determining the section configuration and dimensions of the concrete in the anchoring region.

(5) Regarding to reinforcing methods for concrete in the vicinity of anchorages in post-tensioning techniques, see JSCE "Recommendations for Design and Construction of Prestressed Concrete (1991 edition), Chapter 3 "Anchorage Design".

In pre-tensioning, harmful cracking may occur at the anchorages due to the arrangement and section configuration of tendons, therefore reinforcement must be provided to eliminate adverse effects on member performance.

(6) See present recommendation Part 2, "Construction".

When anchoring devices are embedded in a member after prestressing, the minimum concrete cover for these devices must be ensured.

11.8.2 Minimum reinforcement

(1) The minimum quantity of reinforcement in prestressed concrete shall be 0.1% of the concrete section, where "reinforcement" shall be taken as referring to deformed bars and pre-tensioning tendons.

(2) Reinforcing steel, prestressing steel or CFRM placed in accordance with **11.4.1(3)** above shall have a minimum diameter of 9mm, and shall be spaced not more than 30cm apart.

[COMMENTS]:

(1) Cracking due to shrinkage or temperature gradients may occur in prestressed concrete members prior to prestressing. In order to keep such cracking below harmful levels, all member sections shall include a minimum of 0.1% of the total section area of reinforcing steel, prestressing steel or CFRM. As CFRM has a lower Young’s modulus than steel, placing of an equivalent quantity of CFRM will increase crack widths, but CFRM is also not liable to corrosion, therefore the minimum required quantity of reinforcement has been kept the same both for steel and for CFRM.

For prestressed concrete members in post-tensioning, and for prestressed concrete girders in pre-tensioning, the total quantity of steel and the total quantity of CFRM, including bonding tendons, should be not less than 0.15% of the cross-sectional area of the concrete (c.f. **6.2.2**).

(2) "Prestressing steel or CFRM" refers here to pre-tensioning tendons and grouted post-tensioning tendons.

11.8.3 Joints

It shall be in accordance with JSCE Standard Specification (Design), 10.8.4.