Chapter 9 Performance Verification for Retrofitted structures

9.1 General

The safety, serviceability and restorability of retrofitted structures using the external cable method, bonding and jacketing method and overlaying and jacketing method shall be verified using the methods described in this chapter.

[Commentary]

The performance of retrofitted structures with the selected retrofitting method and construction method must be evaluated using appropriate methods and verified to ensure that performance requirements are fulfilled. The performance of retrofitted structures is accomplished by a combination of the retrofitting members and the members in the existing structure. Accordingly, in the evaluation, not only the performance of the retrofitting members but the cracking and other damage, deformation, and stress in the existing structure must be suitably considered when necessary. This chapter covers the methods used to evaluate the safety, serviceability and restorability of retrofitted structures using the external cable method, bonding and jacketing method and overlaying and jacketing method, with existing technologies. If, due to technological progress, it becomes possible to use even more appropriate performance evaluation methods of confirmed reliability, the methods described here need not necessarily be observed.

9.2 Verification of Safety

9.2.1 Flexural and Axial Load-Carrying Capacity

The safety of the structure shall be verified by confirming that the flexural and axial load-carrying capacity of retrofitted members is greater than the flexural and axial force acting on the structure.

(1) External cable method

(i) The flexural and axial load-carrying capacity of retrofitted member sections or member unit widths shall be calculated in accordance with Section 6.2.1 (2) of the Standard Specification (Design) and the following assumptions.

1. The increase and decrease in fiber strain in the external cables accompanying the increase or decrease of load action can be expressed by the average strain derived from the changes in strain at the external cable center of gravity along the entire external cable length accompanying deformation in the concrete member.
2. The effective depth of the external cable in the area not supported by deviators will change along with deformation of the concrete member.

(ii) The flexural load-carrying capacity of member sections or member unit widths where the effective depth of the external cable is thought to experience relatively little change shall be determined by deriving the tensile stress level of the external cable at the ultimate bending point using **Equation 9.2.1** and calculating based on the assumptions in Section 6.2.1 (2) of the Standard Specification (Design).

$$f_{ps} = f_{pe} + Df_{ps} \quad \text{...}(\textbf{Equation 9.2.1})$$

where:

f_{ps} :Ultimate external cable tensile stress level

f_{pe} :Tensile stress level due to prestressing force

Df_{ps} :Ultimate increase in external cable tensile stress level

(2) Bonding and jacketing method

(i) The axial load-carrying capacity of members retrofitted by jacketing with reinforcing materials shall be determined by methods that give suitable consideration to the effect of the retrofitting members.

(ii) When calculating the load-carrying capacity for members retrofitted by attaching reinforcing materials to the surface to which tensile stress is applied, when these members are subjected to bending moment or bending moment and axial force, peeling shall be considered. If it can be confirmed using appropriate methods that peeling will not occur, the load-carrying capacity may be calculated with no occurrence of peeling.

(iii) If no peeling of the reinforcing materials occurs, the load-carrying capacity for sections or unit widths of members retrofitted by attaching reinforcing materials to the surface to which tensile stress is applied, when these members are subjected to bending moment or bending moment and axial force, shall be calculated in accordance with Section 6.2.1 (2) of the Standard Specification (Design) and the assumptions below, based on the force equilibrium conditions and applicable strain conditions for each type of bending failure mode.

1. The fiber strain for the reinforcing materials is proportional to the distance from the neutral axis of the section.
2. The stress-strain curve for the reinforcing materials is in accordance with 3.4.

(3) Overlaying and jacketing method

Calculation of flexural and axial load-carrying capacity shall be in accordance with Section 6.2 of the Standard Specification (Design).

[Commentary]

(1) (i) Assumptions that a plane remains after deformation cannot be applied to prestressing materials that are not attached to the concrete, such as external cables. Also, since external cables contact concrete members only at the anchorage sections and deviators, an increase in the deformation of the concrete members results in a relatively great change in the effective depth of the external cable in the areas where it is not supported by deviators, and in general the flexural load-carrying capacity tends to be lower than if the effective depth did not change. Here the effect of these factors has been considered and it was decided to use as a basis the calculation of flexural load-carrying capacity of members or member sections retrofitted with external cables.

One specific method of calculating flexural load-carrying capacity is non-linear analysis that takes into consideration the non-linearity and geometric non-linearity of the materials. In recent years, research into the flexural nature of external cable structures has been underway in Japan as well, and technologies are being established for using non-linear analysis to suitably evaluate the above effects and evaluate bending failure capacity with comparative accuracy. However, when calculating the bending failure capacity of members retrofitted with external cables, it would be best to use non-linear analysis which is capable of calculating flexural load-carrying capacity as a structural system.

(ii) When the placement spacing of deviators is comparatively small, as compared to the span length, and deviators are placed near the locations at which maximum and minimum bending moment are applied, etc., the effective depth of the external cables can be seen as experiencing relatively little change even if deformation of the concrete members occurs. In such cases, non-linear analysis is thought to be an advanced method as judged from the general level of technology, so it was decided that **Equation 9.2.1** should be used to calculate the tensile stress of the external cables during failure and to calculate the flexural load-carrying capacity in accordance with conventional bending theory.

The increase in the tensile stress level of external cables at failure is affected by many factors, including the type of external cable, span length and deviator placement status, configuration of deviators, ratio of span and effective depth, and cross-sectional area ratio of internal and external cables. Accordingly, in general the value or the method used to set that value must be specified after the structure to be retrofitted or the structural conditions and other applicable ranges have been identified. However, almost no specific methods have been proposed, and at present the method is to consult standards and research reports relating to external cable configurations to determine whether, to be safe, an increase in stress level should be factored in, and then estimate the value.

According to previous reports[13], when using external cable construction with prestressing steel to build new concrete box girder bridges with a standard span (40 - 60m) and girder height of 2 meters or more, an increase in stress level of about 200 N/mm^2 can be expected even if some leeway is provided. However, it has been pointed out that, when the ratio between girder height and span is 35 or greater, and when the ratio between the placement spacing for deviators and the span is 2/3 or more, caution is required, as in some cases the increase in stress level will be less than 200 N/mm^2. When using this method to calculate the bending failure capacity, it is necessary to consider the bending failure mode of the retrofitted structure. With structures that use both internal and external cables that, like those retrofitted with external cables, have a comparatively large number of internal cables and amount of tension reinforcement compared to the external cables, after yielding of the internal cables, compressive failure of the concrete leading to member failure may occur almost before the stress level of the external cables has increased at all, so this must be considered as well. When a member reaches failure due to flexural compressive failure of the concrete, the distribution of concrete compressive stress levels may be assumed to be the distribution of rectangular compressive stress levels (equivalent stress blocks) in **Figure 6.2.1** in the Standard Specification (Design).

When continuous fiber prestressing materials are used for external cables and one of the continuous fiber prestressing materials reaches ultimate strain and fails, resulting in fiber breakage type bending failure, the result of the compressive force and tensile force at that point must be accurately evaluated to calculate the flexural load-carrying capacity of the retrofitting members. In situations such as when continuous fiber prestressing materials have been placed in many levels, fiber breakage type bending failure will occur when the outermost continuous fiber prestressing materials reach ultimate strain, so in general the flexural load-carrying capacity must be evaluated using the strain of the outermost continuous fiber prestressing materials.

(2) (i) According to past testing, when jacketing with steel plates has been conducted, the axial compressive load-carrying capacity and deformability are increased. When continuous fiber sheets have been used for jacketing, in general there is not much increase in the axial compressive load-carrying capacity while the deformability is greatly increased. It was decided that, when an increase in load-carrying capacity has been confirmed through testing or the like, the effect should be considered using suitable methods.

(ii) (iii) When no peeling of the reinforcing materials occurs, the deformation and load-carrying capacity of the retrofitted members can be determined based on conventional flexural theory for reinforced concrete beams. Conventional bending failure modes include (1) reinforcement yielding - breakage of the continuous fiber reinforcing materials (2) reinforcement yielding - concrete collapse and (3) concrete collapse, so appropriate evaluation is needed. In addition to the conventional bending failure modes, peeling failure is also possible, such as that due to anchorage section failure, or due to peeling at the cracking position that results from tensile force, or due to peeling caused by displacement between the intersections of shear cracking and reinforcing materials. In general, this type of peeling failure should be prevented. However, if the flexural load-carrying capacity in the ultimate peeling state can be appropriately calculated, in some cases no special measures to prevent peeling failure need be taken. To accurately evaluate the behavior of members in the ultimate peeling state, it is necessary to use calculation methods that consider the effect of the occurrence and progress of peeling and numerical analysis methods such as finite element analysis.

Research is currently underway regarding criteria for peeling. However, for the steel plate bonding and continuous fiber sheet bonding methods, the methods shown below based on the anchorage length have been proposed. With the steel plate bonding method, **Equation C9.2.1** can be used to calculate the anchorage length for which steel plate peeling occurs when the steel plate stress level reaches the yield point in the section with the greatest bending moment[14]. In this case, the anchorage length is chosen so the bending moment is a lower value than the section with the greatest bending moment.

$$l_p = a\left\{1-\left(\frac{h_p - x}{h_p x_0}\right)\left(\frac{K_2}{K_1}\right)\left(\frac{I_0}{I}\right)\left(\frac{f_t}{s_{py}}\right)\right\} + K_2 t_p \text{ ....}(\textbf{Equation C9.2.1})$$

where:

- l_p : Anchorage length at which the steel plate peels at the point where it reaches the yield point (mm)
- a : Shear span (mm)
- h_p : Distance from beam compressive edge to steel plate center of gravity

x_0 : Neutral axis position in steel plate bonding section that is effective throughout the entire section (mm)

I_0 : Moment of inertia of cross-section for neutral axis position in steel plate bonding section that is effective throughout the entire section (mm^4)

x : Neutral axis position in steel plate bonding section with tension side ignored (mm)

I : Moment of inertia of cross-section for neutral axis position in steel plate bonding section with tension side ignored (mm^4)

f_t : Tensile strength of concrete (N/mm^2)

s_{py} : Yield point of steel plate (N/mm^2)

t_p : Plate thickness (mm)

K_1, K_2 : Coefficients that consider whether or not there is any cracking in the edge of the steel plate (see **Table C9.2.1**)

Table C9.2.1 Coefficients K_1 and K_2

No cracking at steel plate edge			Cracking at steel plate edge		
Plate thickness (mm)	K_1	K_2	Plate thickness (mm)	K_1	K_2
4.5	1.59	21.5	4.5	0.91	25.4
6.0	1.56	16.9	6.0	0.90	20.4
9.0	1.52	12.1	9.0	0.88	15.2
12.0	1.51	9.3	12.0	0.85	12.1

Equation C9.2.1 was arrived at analytically based on test results in which peeling of the steel plate anchored to the middle of the tension edge of the concrete member occurred at the edge of the steel plate. The equation was validated using several reinforced concrete beam test specimens. Coefficients K_1 and K_2 in the equation were included to consider the effect of whether or not there was any cracking in the edge of the steel plate; as shown in **Table C9.2.1**, these values will vary depending on the thickness of the steel plate. In determining the anchorage length, the value for l_p derived through **Equation C9.2.1** and the length necessary for strengthening should be compared and the longer value made the basic anchorage length. Next, a length equivalent to the effective depth of the member with consideration for moment shift should be added to the basic anchorage length to derive the total anchorage length of the steel plate.

With the continuous fiber sheet bonding method, in general an anchorage that will ensure that peeling does not occur with respect to the design load should be derived using **Equation C9.2.2**.

$$l_{afs} > \frac{f_{afsd} \cdot n \cdot t_{afs}}{t_{afs}} \quad \text{..........(Equation C9.2.2)}$$

where:

l_{afs} : Anchorage length that will ensure that peeling does not occur with respect to the design load (mm)

f_{afsd} : Tensile stress level applied to continuous fiber sheet during design load (N/mm^2)

n_{afs} : Number of ply of continuous fiber sheet

t_{afs} : Thickness of one ply of continuous fiber sheet (mm)

t_{afs} : Bonding strength of continuous fiber sheet to concrete (determined through consideration of the type and number of ply of the continuous fiber sheet, the strength of the concrete surface, member dimensions and other factors) (N/mm^2)

Equation C9.2.1 is a simple method for verifying the occurrence of peeling in members retrofitted with continuous fiber sheets. With this method, the value derived by multiplying the bonding strength by the bond area (bond failure load) can be compared with the tensile force applied to the retrofitted member during flexural resistance, derived based on the assumptions in 9.2.1 (2). The bonding strength will differ depending on the test method, member dimensions, attachment length of continuous fiber sheets, type of continuous fiber sheet and number of ply, concrete surface strength and surface treatment method, and other factors. Accordingly, when using this simple verification method, these factors must be considered to ensure that a reliable value for bonding strength is used. Moreover, this method is not meant to preclude the use of other methods capable of more accurately determining the occurrence of peeling.

(3) Calculations of the flexural load-carrying capacity after flexural retrofitting are based on the premise that the existing members and overlaying concrete behave as a single unit so no peeling occurs. Accordingly, when the overlaying section subjected to bending is in the tension zone, the concrete on the tension side is ignored. In bridge decks with positive moment in which the overlaying section is located in the compression area, the concrete in the overlaying sections and existing sections will have different values for strength and modulus of elasticity, so the characteristic values for overlaying reinforcing materials derived in accordance with Section 3.4.4 should be used to calculate the flexural load-carrying capacity. Nevertheless, in general, the strength of the concrete used in overlaying sections is greater than that of existing sections, so when calculating the flexural load-carrying capacity in such cases, the stress-strain relationship for the existing sections may simply be used for the overlaying sections as well.

9.2.2 Shear Capacity of Bar Members

The safety of the structure shall be verified by confirming that the shear capacity of retrofitted bar members is greater than the shear force applied.

(1) External cable method

The shear capacity of retrofitted bar members V_{ud} shall be derived using **Equation 6.3.2** in the Standard Specification (Design), with consideration given to the reinforcing effect of the external cables, based on the methods below.

1. The shear capacity of bar members that do not use shear reinforcing steel V_{cd} shall be derived using **Equation 6.3.3** in the Standard Specification (Design), with consideration given to the increase in axial compressive force due to the external cables.
2. The components V_{ped} of the effective tensile force of the axial stressing members that are parallel to the shear capacity shall be calculated by adding the components V_{aped} of the effective tensile force of external cables derived with **Equation 9.2.2** that are parallel to shear capacity to those for the internal cables.

$$V_{aped} = P_{aed} \cdot \sin \alpha_{ap} \qquad \textbf{(Equation 9.2.2)}$$

where:

P_{aed} : Effective tensile force of external cables

α_{ap} : Angle formed by external cable and member axis

(2) Bonding and jacketing method

(i) The shear capacity of bar members V_{ud} shall be derived in accordance with 6.3.3 in the Standard Specification (Design), with consideration given to the effect on the shear capacity of the reinforcing materials derived with **Equation 9.2.3**.

$$V_{awd} = b_{aw}\left[A_{aw} f_{awu}\left(\sin a_{aw} + \cos a_{aw}\right)/ b_{aw}\right]z / g_{b} \text{.................(\textbf{Equation 9.2.3})}$$

where:,

V_{awd} : Shear capacity supported by reinforcing materials

b_{aw} : Coefficient indicating stress distribution of reinforcing materials in truss when shear force is applied. In general, this is 1.0 for steel plates and a value less than 1.0 for continuous fiber sheets.

A_{aw} : Cross-sectional area of reinforcing materials per unit width when placed at angle a_{as}

b_{aw} : Unit width of reinforcing materials

f_{awu} : Design yield strength (for steel plate); design tensile strength (for continuous fiber sheet)

a_{aw} : Angle formed by reinforcing materials and member axis

z : Distance from position at which force resulting from compressive stress is applied to the tension steel center of gravity; generally set to d/1.15

g_b : Generally set to 1.15

(ii) Calculations of the shear capacity when reinforcing materials are attached to the side of the bar members shall be derived with methods that give appropriate consideration to the effect of the reinforcing materials.

(3) Overlaying and jacketing method

(i) The shear capacity for retrofitted bar members V_{ud} shall be determined with **Equation 9.2.4** in accordance with 6.3.3 in the Standard Specification (Design), with consideration given to the effect of the overlaying on shear capacity.

$$V_{ud} = V_{cd} + V_{sd} + V_{ped} + V_{asd} \text{ ....(\textbf{Equation 9.2.4})}$$

where:,

V_{cd} :Shear capacity of bar members that do not use shear reinforcing materials utilizing the width and effective depth of the web including the concrete jacketed section

V_{sd} :Shear capacity supported by shear reinforcement in existing members

V_{ped} :Components of axial stressing members with effective tensile force parallel to shear capacity

V_{asd} :Shear capacity supported by added shear reinforcing steel

(ii) The tangential compressive capacity V_{wcd} with respect to the shear capacity of the web concrete shall be determined in accordance with **Equation 6.3.7** in the Standard Specification (Design), using the width and effective depth of the web section including the concrete jacketed section.

[Commentary]

(1) The shear capacity of retrofitted bar members V_{ud} is the sum of the concrete share of the force V_{cd}, the shear reinforcing steel share of the force V_{sd}, the component force in the shear direction V_{ped} of the effective tensile force for the internal cables, and the component force in the shear direction V_{aped} of the effective tensile force for the external cables. This value should be derived with **Equation 6.3.2** in the Standard Specification (Design). In practical terms, as shown in each of the methods (1 and 2), the value for b_n when calculating the concrete share of the force V_{cd}, and the value for component V_{aped} of the effective tensile force of the external cables that is parallel to the shear force, should be used to consider the effect on shear capacity of retrofitting of the external cables.

However, when considering the addition of axial compressive force due to the external cables in calculating b_n, for the sake of simplicity M_0 may be calculated with the entire concrete section considered to be effective, even for sections in which cracking has developed. When calculating the shear direction component V_{ped} for the effective tensile force of the internal cables, consideration must be given to the fact that the effective tensile force of the internal cable will be decreased as a result of retrofitting with external cables. Also, as shown in **Figure C9.2.1**, in cases such as when verification indices are calculated through structural analysis utilizing the converted external force loading method applied to structural models, in which the force resulting from action of external cables on the anchorage and deflection positions is a concentrated load or distributed load, no further consideration should be given to the share of the force V_{aped} from the external cables if the shear force produced by the tension of the external cables has already been considered using some method.

When the increase in external cable tension at the time of shear failure can be evaluated using appropriate methods, the additional tension DP_{aed} can be added to the effective tensile force of the external cables P_{aed} to derive the share of the force V_{aped} from the external cables.

(2) (i) It was decided to express the design shear capacity when bar members are jacketed with reinforcing materials by simply adding the share from the reinforcing materials V_{awd} to the shear capacity of the existing members (the sum of the shares of the concrete and the shear reinforcing materials). According to past test results, the shear capacity supported by continuous fiber sheets when members were jacketed with continuous fiber sheets is lower than the value calculated with truss theory using the tensile strength of the continuous fiber sheets. Reasons for this include the fact that continuous fiber sheets are an elastic body that does not have a plastic range and therefore do not redistribute stress as does steel, so the stress distribution in the truss will not be uniform, and also the fact that, depending on the elongation capacity of the continuous fibers and the quantity used for jacketing, the failure mode is shear compressive failure of the concrete that does not accompany breakage of the continuous fibers. Also, the breakage strain of continuous fiber sheets is greater than the yield strain of conventional reinforcements, so in the event of shear failure it is also possible that excessive diagonal cracking widths, or engagement of the aggregate on the cracking surface, or a drop in the shear force transmitted by the dowel action of the axial reinforcement, may occur, or that local deformation and accumulated stress at the crack location may cause the continuous fiber sheet to break at a lower stress than the uniaxial tensile strength. In these (draft) guidelines, for convenience of design, it was decided to consider the effect of these factors using the coefficient b_{aw} in **Equation 9.2.3**. In other words, the value calculated in accordance with truss theory using tensile strength should be reduced using a coefficient b_{aw} less than 1.0. Accordingly, when using **Equation 9.2.3** to calculate the shear capacity, the coefficient b_{aw} must be set to an appropriate value for the type and quantity of continuous fibers used for jacketing, the member dimensions and so on, based on test results and the like.

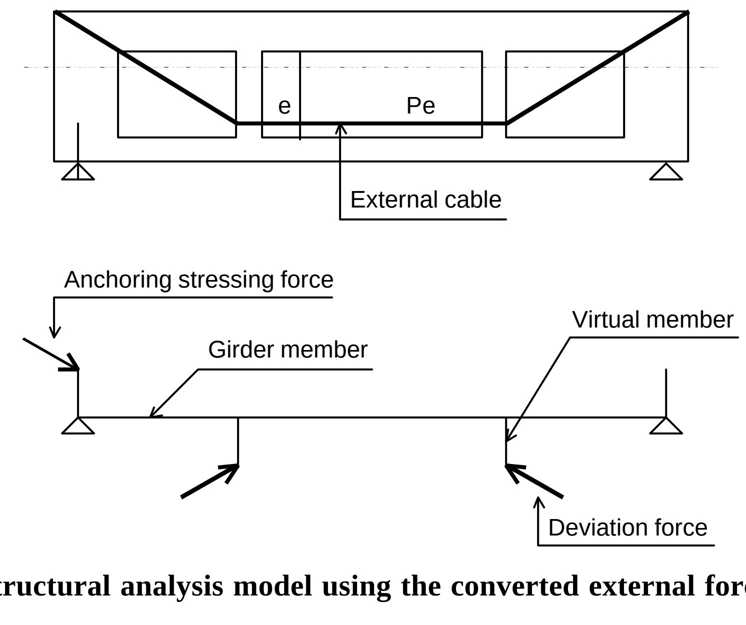

Figure C9.2.1 Structural analysis model using the converted external force loading method

When continuous fiber sheets are used as a reinforcing material, depending on the surface treatment method, the bonding strength of the reinforcing materials and the existing members may not be adequate. In general, when bonding strength does not act, the accumulation of stress at the crack position is less than when a bond exists, making it difficult for the continuous fiber sheet to break. However, this differs in some ways, such as the fact that the shear force supported by the continuous fiber sheet is reduced. When designing without the expectation of bonding strength, an appropriate method that considers these differences must be used to derive the shear capacity.

(ii) When reinforcing girders, in many cases it is not possible to jacket with reinforcing materials. Accordingly, the shear retrofitting method in such cases is to attach the reinforcing materials to the side, or use them to jacket in a U-shape. In such cases, as the ultimate load-carrying capacity is generally determined by the peeling of the reinforcing materials, a great shear reinforcing effect cannot be expected. Accordingly, the reinforcing materials should be anchored mechanically to the top and bottom edges of the girder web, etc. According to previous tests, using anchors to anchor the continuous fiber sheets at the top of the web when jacketing in a U-shape increases the shear capacity. Nevertheless, no methods have been established for rational evaluation of shear capacity, including design methods for anchor anchorages, and when considering this type of shear reinforcement, testing and other appropriate methods must be used to confirm the capacity.

(3) (i) It was decided to represent the shear capacity of retrofitted bar members as the sum of the concrete share V_{cd} including the concrete jacketed sections, the share of the shear reinforcement in the existing members V_{sd}, and the share of the shear reinforcement in the jacketed sections V_{asd}. Here, to the extent that the amount of reinforcement is small, the shear reinforcement in the jacketed sections is thought to have the same effect as the shear reinforcement in the existing members. For this reason, it was thought that truss theory with the angle of the diagonal compressive member set to 45° could basically be applied to the share of shear reinforcement in the jacketed sections V_{sd} as well. In addition, when axial reinforcement is added to the tension side during jacketing, this can be thought of as tension steel when calculating the value for V_{cd}.

9.2.3 Punching Shear Capacity of Surface Members

The safety of the structure shall be verified by confirming that the punching shear capacity of retrofitted surface members is greater than the punching shear force that is applied.

When the overlaying and jacketing method is used and the load surface is separated from the free end of the member or the opening and the eccentricity of the load is small, the punching shear capacity must be calculated with appropriate consideration given to the effect of the reinforcing material.

[Commentary]

The upper surface overlaying method and lower surface overlaying method are used primarily for retrofit of the reinforced concrete decks of highway bridges. A variety of proposals have been made for the punching shear capacity of surface members, but to calculate the punching shear capacity of comparatively thin decks such as the reinforced concrete decks of highway bridges, the punching shear capacity equation[15] based on failure modes derived through testing can be applied, allowing the capacity to be derived with comparative accuracy. It is thought that this equation can also be used to calculate the punching shear capacity of members retrofitted with the upper surface overlaying method and lower surface overlaying method[16][17]. In such cases, design and construction for both overlaying methods must fulfill the considerations in Chapter 8 and both existing and retrofitted sections must form a composite structure that functions as a single unit. The material characteristics of the overlaying sections used for calculation should be values derived through appropriate testing as described in Section 3.4.4. In general, however, as the tensile strength and shear strength of overlaying reinforcing materials are greater than those of existing concrete, in most cases it is safe to use the strength properties of existing concrete to calculate the punching shear capacity. Currently the overlaying for the lower surface overlaying method is thinner than for the upper surface overlaying method and no mechanical construction methods have been established, so peeling of overlaying sections that is dependent on construction accuracy must be considered when evaluating the punching shear capacity.

9.2.4 Flexural Fatigue Capacity

The safety of the structure shall be verified by confirming that the retrofitted member does not experience flexural fatigue failure under load action and environmental action.

(1) External cable method

The flexural fatigue capacity shall be calculated with consideration given to the fatigue performance of the tension reinforcement in the existing sections, the internal cables and the concrete, as well as the fatigue properties of the external cables making up the members. The stress level caused by variable loads that create fatigue shall be calculated based on linear analysis consisting of such methods as direct modeling of external cables as chord members.

(2) Bonding and jacketing method and overlaying and jacketing method

Flexural fatigue capacity must be calculated with appropriate consideration given to the flexural fatigue performance of the existing sections as well as peeling fatigue failure and fatigue properties of reinforcing materials.

[Commentary]

(1) In general, the fatigue capacity of members reinforced with external cables is determined by the fatigue strength of the external cables, tension reinforcements, and internal cables or concrete that make up the members. Here, one method of calculating the stress level caused by variable loads has been shown as an index for verification of the fatigue capacity of members reinforced with external cables. One example of a method in which external cables are directly modeled as chord members is shown in the commentary for Section 9.3.3 (1), so this should be used as reference. When considering the fatigue of the tension reinforcement and internal cables or concrete in existing members, the history of stress levels caused by variable loads that have already been sustained before retrofitting must be considered.

Vibration of external cables is a problem specific to external cables. If the frequency of external cables in areas where they are not supported by deviators is near the frequency of the members, the vibration of the structure produced by the passage of vehicles or the like may cause the external cables themselves to resonate. If this happens, repeated bending stress may act on the external cables, and this can be predicted to reduce the fatigue capacity of the external cables, so the effect of this factor must be confirmed through vibration analysis or other appropriate methods to verify safety.

(2) When calculating the flexural fatigue capacity of bar materials to which reinforcing materials have been attached to the surface to which tension stress is applied, a study of the fatigue of reinforcing materials, steel in existing sections, and concrete, as well as a study of peeling fatigue failure of the reinforcing materials and the retrofitted members, must be performed. Also, when using mechanical anchor anchoring, a study of the fatigue strength of the anchor sections is needed as well. When considering the fatigue of the steel and concrete in existing members, the history of stress levels caused by variable loads that have already been sustained before retrofitting must be considered.

9.2.5 Shear Fatigue Capacity of Bar Members

The safety of the structure shall be verified by confirming that retrofitted bar members do not experience shear fatigue failure under load action and environmental action.

When the bonding and jacketing method is used, the shear fatigue capacity must be calculated with appropriate consideration given to the shear fatigue characteristics of the existing sections as well as the fatigue characteristics and peeling fatigue failure of the reinforcing materials.

[Commentary]

When mechanical anchor anchoring is used, the fatigue strength of the anchor sections must also be studied. When considering the fatigue of the shear reinforcements in the existing sections, the history of stress levels caused by variable loads that have already been sustained before retrofitting must be considered.

9.2.6 Punching Shear Fatigue Capacity of Surface Members

The safety of the structure shall be verified by confirming that surface members do not experience punching shear fatigue failure under load action and environmental action.

When the bonding method and concrete overlaying method is used, the punching shear fatigue capacity must be calculated with appropriate consideration given to the punching shear fatigue characteristics of the existing sections as well as fatigue breakage and peeling fatigue failure of reinforcing materials.

[Commentary]

According to the results of past tests, with the bonding method, attaching reinforcing materials to the surfaces to which tension stress is applied lengthens the fatigue life of surface members. It was decided that appropriate methods must be used to derive values when considering the effect of such retrofitting method. When anchors are anchored mechanically, a study must be made of the fatigue strength of the anchor sections as well.

It has been confirmed that, with the upper surface overlaying method, adding thickness to the deck upper surface through steel fiber reinforced concrete (SFRC) increases fatigue resistance[16]. This is because the steel fibers prevent cracking in the deck upper surface and prevent deterioration caused by the scraping of cracked surfaces against one another. In Reference [16)], the static punching shear capacity equation for decks that have become like girders, with consideration given to the fatigue failure mechanism of actual reinforced concrete decks of highway bridges, in accordance with the punching shear capacity equation in Reference[15], is used to employ a fatigue capacity evaluation equation obtained through wheel load running tests in order to estimate the fatigue capacity of overlaying decks. Here, based on test results, it is thought that no cracking will occur in overlaying sections up to just before final shear failure and that in overlaying sections the entire section is effective. Accordingly, the material characteristics for the overlaying concrete are used for the shear strength of overlaying sections that resist shear force, while the material characteristics for the existing concrete are used for the tensile strength of existing sections that resist tensile force, in order to calculate punching shear capacity and apply it to existing *S-N* curves for reinforced concrete decks that have not been retrofitted and enable the fatigue life of overlaying decks to be estimated.

When considering the punching shear fatigue characteristics of existing sections, the history of stress levels caused by variable loads that have already been sustained before retrofitting must be considered.

9.2.7 External Cable Fretting Fatigue Capacity

The safety of the structure shall be verified by confirming that the external cables do not experience fretting fatigue failure under load action and environmental action.

The fretting fatigue capacity of the external cables must be calculated with consideration given to the stress level due to variable loads as well as the additional bending stress and the bearing stress in the external cables caused by deflection.

[Commentary]

Since stress fluctuations caused by loads are transmitted through anchorage sections or deviators along the entire length of the external cable, due consideration must be given to the fatigue capacity of the external cable at the anchorage sections and deviators. At the deviators in particular, fretting fatigue may be produced in the external cable by the wear, etc. caused by contact between the deflection components and the external cable, and the contact of bare external cables with one another, so a study of safety must be conducted.

In general, the fretting fatigue capacity of external cables tends to decrease as the additional bending stress and the bearing stress produced in the external cable along with deflection increase. Accordingly, these values must be considered to determine the deflection angle and bending radius of the external cables. Particularly in cases such as when there are restrictions on the locations at which the external cables can be placed, the deflection angle of the cables will be greater, so they should be suitably protected against wear, or the amount of stress fluctuation in the external cables must be suitably restricted. The bearing stress should be calculated with the equation (**Equation C1.2.2**) in the retrofitting method manual in Supplement I, in accordance with the bearing stress applied in the direction of the center of the deviator. The additional bending stress should be calculated with **Equation C9.2.1**.

$$s_b = (d \times E)/(2 \times R)$$ ....**(Equation 9.2.1)**

where:

s_b : Additional bending stress (N/mm^2)

d : Bare cable diameter of external cables (mm)

E : Young's modulus of external cables (N/mm^2)

R : Bending radius (mm)

9.2.8 Ultimate Deformation

The safety of the structure shall be verified by confirming that the ultimate deformation of retrofitted members is greater than the response deformation under load action and environmental action.

Ultimate deformation shall be calculated in accordance with the method in Chapter 4 of the Standard Specification (Seismic Design) or other appropriate methods.

Response deformation shall be derived in accordance with the methods in Chapters 3 and 4 of the Standard Specification (Seismic Design).

[Commentary]

The safety of structures is verified by confirming that the members making up the structure will not experience failure. Methods to confirm this include the methods that use member load-bearing capacity described in Sections 9.2.1 through 9.2.7, as well as the method using ultimate deformation. Particularly under seismic action, failure cannot be adequately predicted with load-carrying capacity alone, so ultimate deformation and response deformation must be compared to study whether or not member failure will occur. Ultimate deformation and response deformation can be calculated using the same method as that shown in Section 9.4.1 of these (draft) guidelines. However, the limit value for deformation in Section 9.4.1 is used to consider restorability, so it will be equal to or smaller than the ultimate deformation discussed here. Therefore, when using a value derived in accordance with Section 9.4.1, a value equal to or on the safe side of this value should be used as the ultimate deformation value.

9.3 Verification of Serviceability

9.3.1 Stress Level

The serviceability of the structure shall be verified by comparing the limit value for stress level determined from the serviceability of the structure and the stress level produced by load action and environmental action.

(1) External cable method

The stress level produced in the concrete, reinforcing materials and prestressing materials in retrofitted member sections shall be calculated in accordance with Sections 7.2 and 10.4.1 (1) in the Standard Specification (Design), with consideration given to changes in the structural system before and after retrofitting.

(2) Bonding/jacketing method and overlaying/jacketing method

(i) The stress level produced in concrete and steel in retrofitted member sections shall be calculated in accordance with Section 7.2 in the Standard Specification (Design).

(ii) The permanent load applied to the structure since before retrofitting shall be calculated as the stress level in existing sections, while the increase in permanent load and variable loads after retrofitting shall be determined by calculating the stress level in the composite section formed by existing and retrofitted sections, and these values shall be together.

[Commentary]

(1) The stress level of retrofitted members is calculated as the sum of the stress level caused by permanent loads applied since before retrofitting and the stress level caused by permanent loads and variable loads applied after retrofitting.

The stress level caused by permanent loads applied before retrofitting should be calculated in accordance with Sections 7.2 and 10.4.1 in the Standard Specification (Design), with appropriate consideration given to the cracking status. The stress level caused by permanent loads and variable loads applied after retrofitting should be calculated in accordance with assumptions 1 - 4 below, with the effect of the external cables considered.

1 The prestressing force of the external cables remains constant even if the applied force increases or decreases.

2 The effective depth of the external cables remains constant even if the concrete members are deformed.

3 External cables are an elastic body.

4 External cables are not considered in effective sections.

Assumptions 1 and 2 relate to the changes in effective depth of the external cables accompanying the increase in external cable strain and the deformation of concrete members. In verifying serviceability, since member deformation is generally thought to be in the minute deformation range, it was decided that the increase in external cable strain and change in effective depth caused by concrete member deformation need not be considered. Accordingly, the concrete stress level may be derived by ignoring the increased stress of external cables and assuming that eccentric axial force caused by effective prestressing force is being applied.

Assumption 3 was established because, when the tension stress level of the stressing members exceeds the elastic limit, a variety of problems occur, such as that structural analysis and the assumptions regarding the calculation of stress levels do not apply, and prestressing force can no longer be treated as external force. However, in the case of retrofitting, it is thought that at times the stress level acting on concrete, steel and internal cables will exceed the elastic limit stress level. In such cases, the stress level must be calculated with a method that can suitably evaluate the effect of this factor.

Assumption 4 was established because, in general, the impact of external cables on effective sections is thought to be comparatively minor. Accordingly, when the external cable method is not used in combination with other retrofitting methods such as the bonding and jacketing method, the existing sections may basically be thought of as the effective sections, even after retrofitting.

When retrofitting reinforced concrete members with external cables, and when retrofitting prestressed concrete members that permit cracking during the service life after retrofitting, the effect of relaxation of the prestressing cable, concrete creep, the effect of shrinkage and the effect of reinforcement constraints must be considered to calculate the stress level of the concrete and steel under permanent loads, as noted in the provisions regarding polymer reinforced concrete (PRC) structures in Section 10.4.1 of the Standard Specification (Design). Also, with prestressed concrete members, the fact that the effective tensile force of the internal cables will be reduced must also be considered.

(2) The stress level restrictions in Section 7.3 of the Standard Specification (Design) are recommended as restrictions on the stress level under general load action.

When continuous fiber sheets are used, calculations of the stress level must consider the orientation of the reinforcing materials. When the reinforcing materials and the existing sections are bonded together, calculations of the stress level produced in the concrete, steel and reinforcing materials in member sections should be based on the following assumptions.

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

2 Concrete, steel and reinforcing materials are elastic bodies.

3 The tension stress of the concrete is ignored.

4 As a rule, the stress-strain curve for the concrete and steel is in accordance with Chapter 3 of the Standard Specification (Design).

5 The Young's modulus of the reinforcing materials is in accordance with Section 3.4 of these (draft) guidelines.

When bonding strength is not thought to exist between the reinforcing materials and the concrete, appropriate methods must be used to calculate their individual stress levels.

9.3.2 Crack Width

The serviceability of the structure shall be verified by comparing the restrictions on crack width established from structure serviceability and the width of cracking produced under load action and environmental action.

(1) External cable method

The width of flexural cracking in retrofitted structures shall be calculated in accordance with Section 7.4.4 in the Standard Specification (Design), with consideration given to the effect of prestressing by external cables.

(2) Bonding/jacketing method and overlaying/jacketing method

The effect of the reinforcing materials shall be considered when calculating flexural cracking widths.

[Commentary]

(1) When cracking during the service life after retrofitting is permitted, the flexural cracking width should be calculated for the tension reinforcements or internal cables at the location closest to the existing member concrete surface, using the normal flexural cracking calculation Equation shown in **Equation 7.4.1** in the Standard Specification (Design).

Values s_{se} and s_{pe} for the increase in steel stress level, shown in **Equation 7.4.1**, must be derived with consideration for both the stress status of the concrete and steel before and after retrofitting and the status of the occurrence of cracking. In other words, in member sections in which cracking has occurred before retrofitting, the value for increase s_{se} or s_{pe} is the sum of (1) the value for the steel stress level that is applied before retrofitting, minus that portion of the steel stress level when the concrete at the same location as the steel changes from compression to zero, and (2) the steel stress level caused by the load applied after retrofitting. In member sections in which cracking occurs after retrofitting, the value for the steel stress level due to load that is applied after retrofitting, minus that portion of the steel stress level when the concrete changes from compression to zero, is the value for increase s_{se} or s_{pe}.

(2) If the spacing of cracks in members to which reinforcing materials have been attached is the same as that in reinforced concrete members, the reinforcement stress level, determined through consideration for the effect of the reinforcing materials, may be used to evaluate a value on the safe side for the width of cracks in retrofitted members, in accordance with **Equation 7.4.1** in the Standard Specification (Design).

In general, cracking in members to which reinforcing materials have been attached is more widely distributed than that in members without reinforcing materials attached, and as a result the crack width is reduced. In uniaxial tensile tests of members reinforced with carbon fiber sheets, the crack width was almost exactly proportional to the average strain of the carbon fiber sheet and reinforcement, and was 0.3 - 0.7 times that of unretrofitted members just before the stage of reinforcement yield[18][19]. Accordingly, **Equation C9.3.1**, in which the crack width indicated by **Equation 7.4.1** in the Standard Specification (Design) is multiplied by the maximum crack ratio of 0.7, may be used to determine the flexural cracking width when the lower surfaces of beams have been reinforced with continuous fiber sheets.

$$w = 0.7k\left[4c + 0.7(c_s - f)\right]\left[\frac{s_{se}}{E_s}\left(\text{or } \frac{s_{pe}}{E_p}\right) + e'_{cs}\right] \text{..................}(\textbf{Equation C9.3.1})$$

Since there is little data on cases in which steel plates have been used to reinforce the lower surfaces of flexural members, appropriate methods should be used to study the cracking width. Also, the mechanism by which shear cracking occurs and progresses differs from that of flexural cracking, so this should be studied using appropriate methods set forth elsewhere. In addition, studies of crack widths are not needed when reinforcing materials have been attached to the surfaces of concrete structures, since these surfaces have been protected.

When the overlaying and jacketing method has been used, the stress level derived for the composite section with the existing section and overlaying section bonded together may be used to evaluate the crack width in accordance with **Equation 7.4.1** in the Standard Specification (Design).

9.3.3 Displacement and Deformation

The serviceability of the structure shall be verified by comparing the restrictions on displacement and deformation determined from structure serviceability and the amount of displacement and deformation produced under load action and environmental action.

(1) External cable method

The displacement and deformation of retrofitted members shall be calculated as the sum of the displacement and deformation resulting from permanent loads applied before retrofitting, and the displacement and deformation resulting from permanent loads and variable loads applied after retrofitting. The displacement and deformation before and after displacement shall be calculated based on the methods shown below.

(i) Calculation of the displacement and deformation before retrofitting shall be in accordance with Section 7.5.3 in the Standard Specification (Design).

(ii) Calculation of the displacement and deformation after retrofitting shall be calculated in accordance with linear analysis, using methods such as direct modeling of external cables as chord members. As a rule, the decrease in rigidity due to cracking shall be considered when calculating the rigidity of concrete members used for analysis.

(2) Bonding/jacketing method and overlaying/jacketing method

Calculation of the displacement and deformation of retrofitted members shall be in accordance with Section 7.5.3 in the Standard Specification (Design).

[Commentary]

(1) The displacement and deformation due to permanent loads applied before retrofitting should be calculated in accordance with Section 7.5.3 in the Standard Specification (Design), with appropriate consideration given to the effect of decreased rigidity due to cracking and the progress of concrete creeping and drying shrinkage. However, when the variable loads applied before retrofitting are greater than the permanent loads, and the continuity and frequency of the variable loads are high, the effect of the history of variable loads must be considered in addition to the displacement and deformation due to permanent loads when calculating the displacement and deformation before retrofitting.

It was decided that the displacement and deformation of members reinforced with external cables should be calculated using linear analysis, through methods such as member evaluation with external cables directly modeled as chord members. As shown in **Figure C9.3.1**, the member evaluation methods involve applying variable loads to frame models with external cables modeled as chord members, and beams and virtual column members modeled as bar members, and directly deriving the variable stress level, etc. of the external cables. When the structure in question can be modeled as a two-dimensional model, this method enables calculation of the variable stress with comparative ease. However, modeling must appropriately model factors such as the characteristics of friction between the deflection components and the external cables.

When in actual practice it is not necessary to determine the precise displacement and deformation, and when it has been determined that no cracking will occur after retrofitting through calculations with the entire section effective, even members in which cracking has occurred before retrofitting can be considered members in which no cracking will occur, and the analysis can be conducted using the moment of inertia with the entire section effective.

(2) The displacement and deformation of retrofitted members must be calculated through evaluation with appropriate rigidity of the status of cracking in existing members. If the width of cracks occurring in existing members is great, generally the cracks are filled as surface treatment before retrofitting. The effect of this factor must be considered when calculating the displacement and deformation of retrofitted members.

With the lower surface overlaying method, the thickness is added on the tension side of existing members, so existing cracks are constrained and, until cracking occurs in the sections after retrofitting, the retrofitted sections appear to exhibit rigid section behavior and function with the entire section effective. After that point, as the load increases, the rigidity of the retrofitted section approaches the behavior of reinforced concrete sections in which the tension side concrete is ignored, and so the equation (**Equation C7.5.7**) in the Standard Specification (Design) holds true for section rigidity after lower surface thickness addition as well. Accordingly, it was thought that, when calculating displacement and deformation, this equation can be used for calculation by performing numerical integration in the axial direction, and so it was decided that the process should be in accordance with Section 7.5.3 in the Standard Specification (Design).

In general, if it has only been a few years since the structure was built, it is possible that additional displacement and deformation will occur due to concrete shrinkage, creep and so on. In such cases, the additional displacement and deformation must be appropriately evaluated and added.

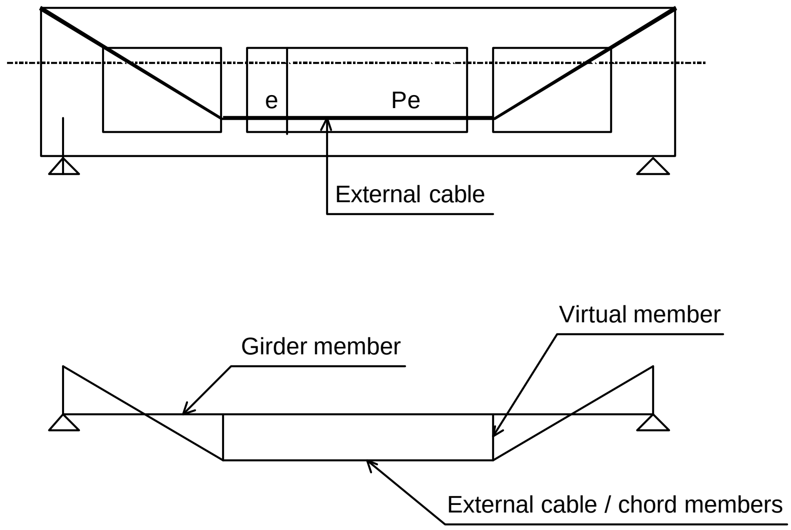

Figure C9.3.1 Structural analysis model using member evaluation method

9.4 Verification of Restorability

9.4.1 Deformation

The restorability of the structure shall be verified by confirming that the restrictions on deformation determined from the restorability of retrofitted members are greater than the response deformation under load action and environmental action. Here deformation shall refer to major deformation when load is applied or residual deformation after load application.

The restrictions on deformation determined from the restorability of retrofitted members shall be established in accordance with the method in Chapter 4 of the Standard Specification (Seismic Design) or other appropriate methods.

The response displacement shall be determined in accordance with the methods in Chapter 3 and Chapter 4 of the Standard Specification (Seismic Design).

[Commentary]

In calculating the response displacement, ductility ratio and residual displacement of the structure with respect to an earthquake, the structure must be appropriately expressed by means of a model, and the material characteristics must be modeled such that its dynamic properties are recreated. If no failure criteria have been incorporated into the analysis model, the failure mode of the constituent materials must be determined; flexural load-carrying capacity and shear capacity should be calculated in accordance with Sections 6.2.1 and 6.3.3 of the Standard Specification (Design) if the member is a non-retrofitted member and in accordance with Sections 9.2.1 and 9.2.2 of these (draft) guidelines if the member is a retrofitted member.

In domains with members where the use of plastic hinges is assumed, the deformation performance of structures is ensured by placing the reinforcing materials at right angles to the member axis and jacketing if many are used. For design evaluations, the method of applying ductility as a function of the flexural shear capacity ratio of the member in the plastic hinge domain, and the method of introducing the stress-strain relationship of concrete restrained in the plastic hinge domain to calculate the ultimate flexural deformation, are used; the method is selected to match the characteristics of the structure (size, type, etc.). The retrofitting methods covered in these (draft) guidelines also use similar retrofitting methods; at present, the evaluation method for the former method is primarily member testing with the flexural shear capacity ratio as a parameter, while for the latter an evaluation method is being constructed through the introduction of the stress-strain relationship by means of axial compression tests.

In the commentary for Section 4.2 of the Standard Specification (Seismic Design), ductility is provided by **Equation C4.2.2** in the range of $V_{cd}/V_{mu} \leq 1.4$ and $V_{sd}/V_{mu} \leq 1.4$. For $V_{yd}/V_{mu} \geq 2.0$, the failure mode is stable bending failure, and for ordinary members having axial compression stress of approximately 1.0 N/mm^2, concrete stress of 20 - 30 N/mm^2 and reinforcement yield strength of 300 - 400 N/mm^2, the ductility will be approximately 10. In material tests, when the ductility exceeds approximately 10, an increase in the amount of shear reinforcement tends not to have a notable effect on increased ductility, resulting in cases such as a history of repeated loads causing the reinforcement to break due to low cycle fatigue[20][21]. In the case of retrofitted columns as well, as shown by the example in **Figure C9.4.1**, in each of the test results[22][23] for steel plate jacketing and reinforced concrete jacketing (using PCa molds with steel fibers) and carbon fiber sheet retrofitted columns, conspicuous retrofitting effectiveness is apparent up to member angles of approximately 0.05.

The effect of ductility retrofitting differs depending on the characteristics of the retrofitting method, and the calculation equations are also different. Particularly with methods that use materials for which existing shear capacity equations cannot be applied, the characteristics of the method will be related to shear capacity equations and ductility calculation equations, so it is important to note that the form will differ from the **Equation C4.2.2** in the Standard Specification (Seismic Design). Also, as can be seen in the examples in **Figure C9.4.1** using steel plates and the PCa mold with steel fibers, when reinforcing materials that have great flexural tension ductility are used continuously in the member axial direction, the effect of preventing inner concrete peeling in the plastic hinge domains is even more apparent above the maximum load-carrying capacity, and it is should also be noted that in some cases the ultimate member angle will greatly exceed 0.05.

The following is an outline of the results of research for railway structures and highway structures regarding methods used to calculate deformation performance (or ductility) with jacketing methods. With railway structures, in each case ductility is provided in the form of an empirical formula for evaluating, on the safe side, the results of an alternating positive-negative load test using a mock-up column test piece[22][25-29]. Here, the equation used to calculate ductility for the jacketing methods using steel plates and concrete materials is one that uses functions for flexural shear capacity ratio and shear reinforcement steel ratio; in contrast, in the case of jacketing with new materials and other reinforcing materials, the equation is provided by only a function for flexural shear capacity ratio. In the case of highway structures, depending on the differences between retrofitting methods, a stress-strain curve [30][31] showing the side constraining effect of the concrete is applied and a fiber model is used to provide ultimate displacement.

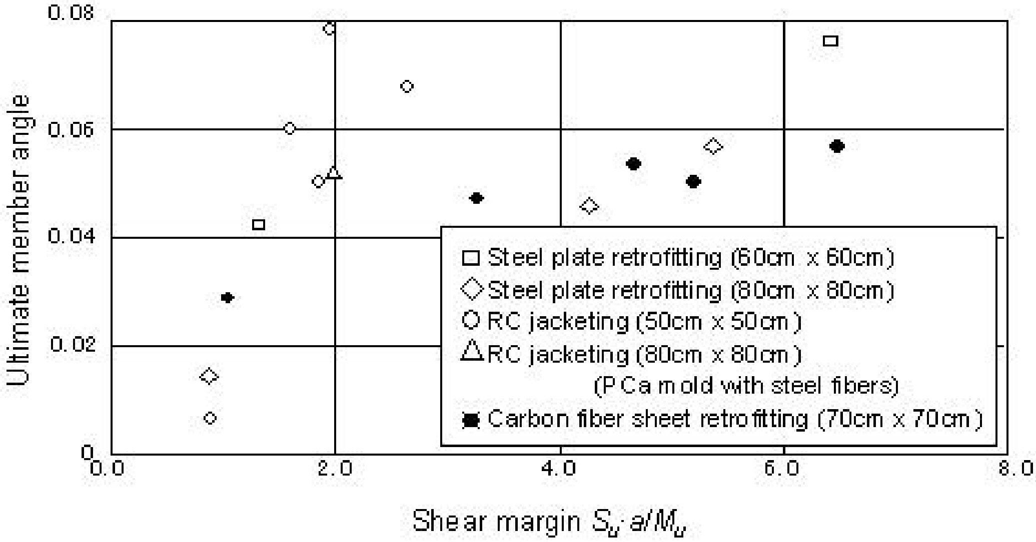

Figure C9.4.1 Example of the relationship between ultimate member angle and shear margin[24]

As an analytical approach to deformation performance of existing or retrofitted columns, it is thought that flexural deformation analysis using fiber models or methods using two-dimensional or three-dimensional non-linear finite element analysis can be used. Considerations that have been pointed out for the former analysis method using fiber models are the need for a stress-strain relationship exceeding the maximum stress of the concrete, a reinforcement stress-strain relationship that takes into account peeling and buckling of the concrete covering, and methods for accurate evaluation of member rotation due to the plastic hinge length and slippage[32]. Also, it has been reported that analysis using the finite element method (FEM), based on appropriate constitutive models and algorithms, enables direct evaluation of not only the occurrence and repetition of slipping and diagonal cracking in the reinforcement and reduction of shear resistance of the concrete due to major deformation, but also the effect of size in the tension softening domain, thus enabling accurate analysis of columns that experience stress failure after flexural yielding[33]. In addition, when three-dimensional structural laws are used, reflection of a concrete constraint effect due to the use of side reinforcing materials can also be anticipated[34].

In principle, it is possible to greatly reduce residual displacement by introducing prestressing in the outer axial direction of column members, and research is already being conducted to evaluate the results of introducing prestressing in the interior of sections on newly constructed bridge piers and the like[35][36]. Along with progress in future research relating to resilience capacity, energy absorption and deformation performance, a close watch should be kept on the applicability of the external cable method to seismic retrofitting of column members.

9.4.2 Stress Level

The restorability of the structure shall be verified by confirming that the restrictions on stress level determined from the restorability of retrofitted members are greater than the stress level under load action and environmental action.

(1) External cable method

The stress level of retrofitted members shall be calculated as the sum of the stress level caused by permanent loads applied before retrofitting and the stress level of permanent loads and accidental loads applied after retrofitting. The stress levels before and after retrofitting shall be calculated based on Section 9.3.1.

(2) Bonding/jacketing method and overlaying/jacketing method

The stress level for the permanent load applied since before retrofitting shall be calculated as the stress level in existing sections, while the stress level for the permanent load and accidental loads added after retrofitting shall be calculated for the composite section made up of the existing sections and retrofitted sections, and these values shall be added together.

[Commentary]

When the structure has sustained damage as the result of an earthquake or the like, if the residual displacement is in a small enough range, restoration of the structure's performance will be easy. The stress level of the reinforcement and tendon is generally in the elastic range, and if it can be determined that the structure is sufficiently safe with respect to concrete compressive damage, the residual displacement can be thought of as sufficiently minor. For this reason, it was decided that the stress level may be calculated as an index for verifying restorability. In such cases, the analysis model should be in accordance with the Standard Specification (Seismic Design). Also, calculation of the stress level should be done in accordance with Section 9.3.1 in these (draft) guidelines.

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