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RECENT TRENDS IN PRESTRESSED CONCRETE STRUCTURES WITH EXTERNAL CABLES

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External prestressing is coming into wide use, and recently the number of prestressed concrete bridges using external cables (tendons) has been increasing remarkably. External prestressing has been used to rehabilitate damaged concrete structures as well as for new ones. In this paper, the structural behavior of externally prestressed concrete beam members and the strength of external tendon anchorages and deviators, as well as analytical methods for them, are reviewed. Some typical examples of the rehabilitation of damaged concrete bridges are also covered. In addition, several major structural design provisions for external prestressing are described.

Keywords : *external prestressing, structural behavior, analysis, design, rehabilitation*

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1. INTRODUCTION

External prestressing is now widely used in many countries, and a considerable number of prestressed concrete bridges have been constructed with external cables (tendons). This technique was adopted in bridge construction during the 1920s and 1950s in Europe. Later, however, external prestressing was almost completely abandoned for lack of ways to prevent corrosion of the tendons and the high cost of their repair. After external prestressing was adopted as a means of strengthening prestressed bridges in France during the 1970s, however, the protective systems for tendon corrosion developed markedly[1]. With construction of the Florida Key bridges with external tendons in the U.S.A, the method grew rapidly[2]. The use of external tendons alone or in combination with internal tendons has been promoted for bridge construction in France in particular. A systematic theory of external prestressing was formulated in 1983[3], and the first draft of design standards[4] was issued in 1990 by SETRA.

In Japan, since the combined use of external and internal tendons in the Sasamegawa bridge on the tohokushinkansen line in 1985, external prestressing has been increasingly used[5].

Recently, precast concrete segments have come into common use for large-scale structures, and segmental bridges using external tendons have often been reported[6]. External tendon systems can also be used efficiently to repair or strengthen damaged concrete structures because construction is easy and the effects are highly reliable.

Although these various advantages make external prestressing very attractive, there still remain certain problems that must be examined in greater detail[1],[7],[8].

This paper reviews the state-of-the-art as regards the fundamental structural properties of externally prestressed concrete members and methods of analysing them. Some inherent problems associated with the external tendon system are also described, and application of the method to the repair or strengthening of damaged concrete structures is explained. Prevalent design specifications are also mentioned.

2. BEHAVIOR AND ANALYSIS OF EXTERNALLY PRESTRESSED CONCRETE MEMBERS

2.1 Fundamental Behavior of Externally Prestressed Members

During the past ten years, a number of experimental and theoretical results have been reported as regards externally prestressed concrete beam members. Studies of monolithically cast concrete beams are the focus here.

(1) Simply supported beams

The behavior of simply supported beams with external tendons is influenced by various factors such as the spacing between deviators, the total number of tendons, the amount of ordinary nonprestressing steel, the prestressing force, the ratio of span to effective depth, the load pattern, and the mix of external and internal tendons, etc..

According to many test results, changes in tendon eccentricity, namely, the secondary effects of deformation under the applied load, markedly influences the flexural behavior of externally prestressed beams. If the spacing betweeen deviators is increased, the ultimate flexural strength is reduced[9],[10],[11], and plastic deformability falls[9],[12]. This is because, in externally prestressed beams, the compressive strain of the concrete becomes critical at the ultimate state rather before the tensile strain of the tendons due to compatibility condition of deformation, resulting in compressive failure and concrete crushing. Thus externally prestressed beams are likely to fail in over-reinforced flexural mode, while while internally prestressed beams with bonded tendons fail in under-reinforced mode[13].

The behavior of externally prestressed beams depends on their dimensions. With increasing span/

effective depth ratio, the increase in external tendon stress becomes small, resulting in lower ultimate flexural strength and less absorbed energy up to failure. This tendency is more notable in beams without deviators[14].

The ultimate moment of resistance is affected greatly by the loading conditions in the case of externally prestressed beams, while it is almost independent in the case of internally prestressed ones with bonded tendons[11]. In the former, the ultimate moment of resistance falls with decreasing distance between the two loads when subjected to symmetrical two-point loading for a given span length, and the resisting moment under a center point loading often falls down to the calculated value in which the initial prestressing force is regarded as an external force, neglecting the stress increase in external tendons[15].

Externally prestressed beams in which one of the deviators is installed at mid-span exhibit similar flexural behavior to internally unbonded prestressed beams[9]. Further, a modified type of externally prestressed beam, in which second-stage concrete is cast on top of the bottom flange to cover the external ducts for bond development, behaves in a similar manner to an internally unbonded one[16]. Incidentally, the effect of deviators becomes much more marked when deflection is large in beams with a relatively small number of tendons[14].

A PRC (prestressed reinforced concrete) section with an appropriate amount of nonprestressed steel effectively improves the distribution of flexural cracks and the ultimate deformability of externally prestressed beams[11],[14],[17], and this type PRC is in practical use[18].

Test results[9],[19] on beams with mixed internal and external tendons indicate that the distribution of cracks, ultimate strength, and ductility are improved as compared with beams having external tendons alone. Such use of mixed tendons is common for structures in which the stresses due to construction loads and in-service loads are carried by internal tendons and external tendons, respectively.

On the other hand, studies on behavior under shear have so far been very few compared to that under flexure.

According to tests on PRC beams using external tendons without web reinforcement[20], measured shear strength is larger than that calculated according to the JSCE Standards[21], the AIJ Provisions[22], and the ACI Code[23], in all of which the safety factor is set to 1.0. This tendency to underestimate is particularly marked when the introduced prestress is high. However, the difference in shear strength between beams of external tendon type and internal tendon type was not clarified in these studies. According to other shear tests[24] in which ordinary reinforced concrete(RC) beams with a small amount of web reinforcement were externally prestressed, the measured shear cracking strength agrees roughly with the calculations. However, the measured ultimate shear strength is considerably larger than that calculated based on the JSCE design equation[21], in which the effect of prestress is considered in terms of both the decompression moment and the vertical component of prestressing force in the shear resistance carried by the concrete, V_c. Further, test results[25] on precast segmental T-shaped beams indicate that no diagonal cracks occur in externally prestressed beams, although the web thickness was thinner by 40% than that in each corresponding internally prestressed beam, demonstrating that external prestressing is very useful for increasing shear strength.

As for members with unbonded longitudinal steel, such as external tendons, the effects of the lack of steel bonding on failure mode, strength, and arch formation were studied by comparing the behavior of simple RC beams with and without bonding of deformed bars in regions except the anchorage zone beyond the supports[26]. According to tests on the unbonded beams, except in cases where the shear span to effective depth ratio was less than 2.0, no diagonal cracks appeared, shear failure never occurred even without web reinforcement, and the load carrying capacity of residual arch action increased markedly due to formation of a tight arch rib. These findings are very useful as basic information on the shear behavior of externally prestressed beams. Incidentally, it has also been reported[27] that the truss model can be usefully applied to predict the shear strength of unbonded prestressed beams, and the design shear equations for bonded members may be adopted irrespective of the shear failure mode when nonprestressed steel is properly arranged.

(2) Continuous beams

The results of tests on a ⅓-scale model of a continuous concrete slab bridge with two spans of 30m have been reported[28],[29]. In this case, the cable system was a mix of internal and external tendons. The latter were coupled into the slab over an intermediate support, and placed outside the cross section below the slab in other regions. The required amount of internal prestressing entirely within the slab was determined so as to resist the dead load during construction. Results showed that the stress increase in external tendons was about 8% of the yield strength under all service loads, and no signs of distress occurred after 2×10^6 cycles at a stress amplitude equal to ½ of the service load. From these dynamic tests, the impact factor was found to be about 20% at most, or nearly equal to that of conventional bridges in the same span range. Thus, as shown in Fig.1, the tested continuous slab behaved in a ductile manner with yielding of the external tendons.

Some tests have been conducted on two-span continuous beams with two distinctive external tendon arrangements[30]: one of ordinary type and the other of innovative type where large eccentric external tendons are installed outside of the girder height at the intermediate support and at midspan. As shown in Fig.2, in the ordinary type, a moment redistribution began after flexural cracking, and the ratio of measured intermediate support moment to elastic calculation was about 0.9 in the ultimate state, implying an approximately 10% redistribution of moment. The beam of innovative type behaved in a similar manner. In cases where the load pattern or tendon profile is such that the distribution of concrete strain at the level of the external tendons is relatively uniform along the entire beam length without local protrusion, the increase in external tendon stress in the ultimate state was large[31]. For a given prestress in a tensile fiber in the beam section, the innovative type beam with fewer external tendons had nearly the same ultimate strength and ductility as the ordinary type, resulting in better economy[30].

As for moment redistribution, which is a very important characteristic in statically indeterminate structures, a similar to the above has been confirmed in other tests[32] on externally prestressed continuous beams. Meanwhile, a similarly marked moment redistribution occurs at the ultimate state in externally prestressed segmental continuous beams, resulting in the formation of plastic hinges at critical sections[33]. Other tests on segmental beams have shown similar behavior[34],[35].

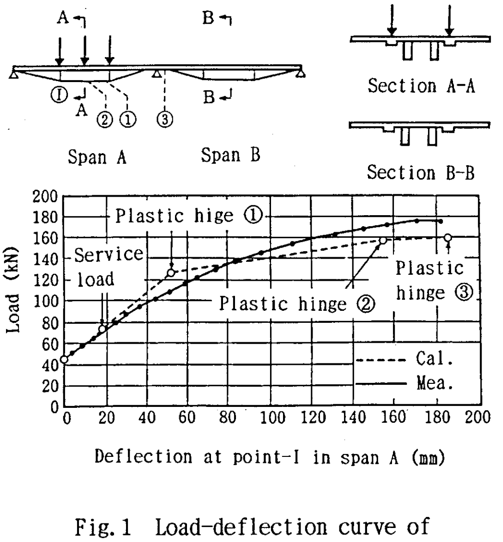


Fig.1 Load-deflection curve of continuous slab model[29]

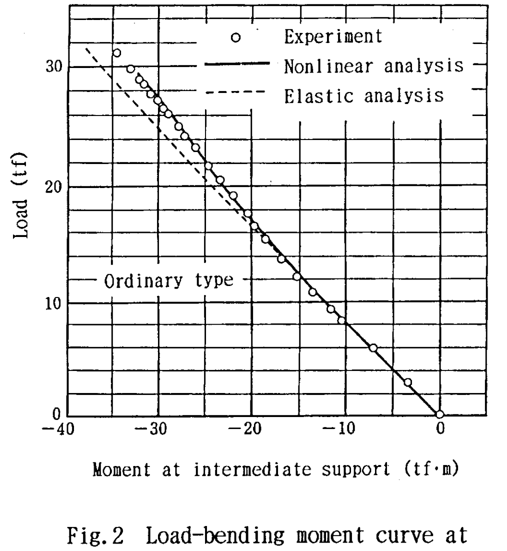


Fig.2 Load-bending moment curve at intermediate support[30]

The above mentioned continuous beams, except those in references [30], were prestressed with a mix of external and internal tendons. According to the experiments on simply supported beams mentioned above and the analytical results[38] mentioned later, the plastic deformability of beams with external tendons alone is less than that of ones with mixed external and internal tendons or internal tendons alone. Therefore, the degree of moment redistrubution, the formation of plastic hinges, and the ultimate strength of the externally prestressed continuous beams needs to be evaluated in consideration of the ratio of external to internal tendons.

2.2 Behavior of Segmental Beams

In precast segmental members with external tendons, it is very important to examine how the discontinuities in ordinary reinforcing steel and local rotation due to crack opening at joints between segments affect behavior.

Some test results focusing on the joints between externally prestressed segmental beams are described in the following.

According to the tests on simply supported beams, in which joint types and segment lengths were adopted as the main test variables, cracking occurred in concrete adjacent to the joints where the joints were epoxied, while dry joints opened[36]. As shown in Fig.3, the cracking load corresponding to an abrupt change in the initial slope of the load vs. deflection curve is somewhat larger in beams with epoxy joints than in dry jointed beams, but no significant difference can be observed in overall flexural behavior between these two beams. The reason for the maximum strength of the monolithic beam being considerably greater than that of the two segment beams is that the steel is continuous over the entire beam length. The flexural behavior in the ultimate state was found to be influenced by segment length, because the number of cracks affecting the stress concentration in the compressive zone of the section was dependent upon length.

A test has also been done to examine the effect of joint type (dry joint and epoxy joint) on the behavior of a three-span segmental box-section beam with external tendons[35]. The results revealed that cracking occurred in concrete adjacent to the joints in case of the epoxy joint, and the cracking strength of the epoxy joint was considerably greater than that of the dry joint. However, the ultimate flexural strength was scarcely influenced by the joint type, being about 6.0 times the equivalent AASHTO HS-20 truck load. A marked redistribution of moment was

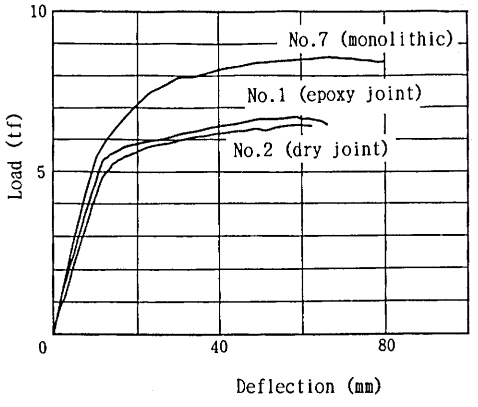


Fig.3 Load-deflection curves of simple beams[36]

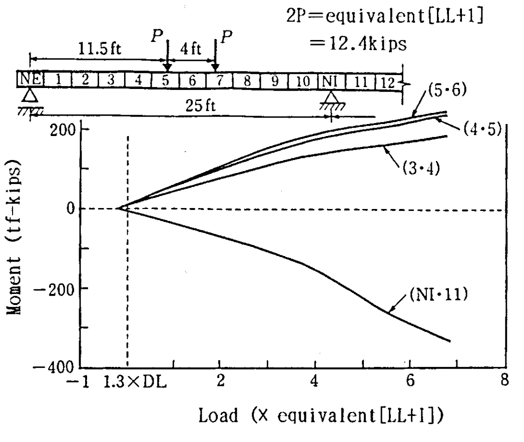


Fig.4 Load-bending moment of continuous beam[34]

observed in the ultimate state due to a large concentrated rotation at the joints in both types, as shown in Fig.4. This figure shows the case of the dry joint, where the notation (5,6) for example represents the joint between segments 5 and 6. It is also recognized that the intensity of the shear force transmitted across the joint governs the mode of crack occurrence (flexural cracking and diagonal cracking modes), which significantly affected the mechanism of local rotation at the joint.

Further, according to similar tests[37] on a three-span continuous segmental box-section beam, joint opening at concrete crushing was larger in epoxy joints than in dry ones, and the ultimate strength and the ductility were substantially higher in the former (Fig.5). This is due to a difference in cracking pattern between these two types of joint. As shown in Fig.6, in the epoxy joint the cracks extend transversely over the bottom of the upper flange at three locations, which implies that an increase in volume of the highly stressed and strained concrete causes higher compressive deformation before crushing. In contrast, only one crack occurs in the case of the dry joint, which prevents further cracking from occurring and limits the maximum compressive zone to the joint area.

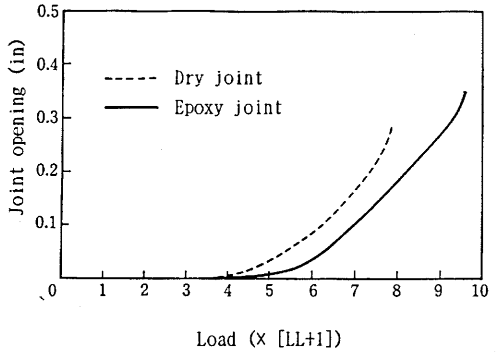


Fig.5 Relationship between load and maximum opening width at joint[37]

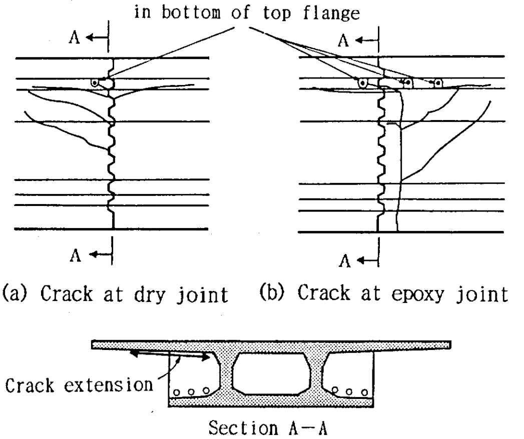


Fig.6 Crack patterns at joint[37]

2.3 Analytical Methods

(1) Basic analytical method

Since external tendons are unbonded with the concrete, conventional analysis of flexure based on compatibility between strains in the tendons and the concrete at a particular section is not applicable to externally prestressed beams. Typical analytical methods for these beam members are described below.

i) Rigorous Method

This is a method in which the behavior of externally prestressed beam is analyzed by a trial-and-error procedure using the conventional analysis of flexure, taking into account both the compatibility of deformation over the entire beam length and the change in tendon eccentricity with increasing beam deformation.

This method provides a solutuion with high accuracy, but a highly customized program must be used for different structural systems, such as simple beams and continuous beams.

Mutuyoshi et al.[10] analyzed a number of simply supported externally prestressed beams using this method, and proposed two reduction factors related to effective depth and external tendon strain for use in practical design. They also confirmed that the ultimate flexural strength of externally prestressed beams could be predicted by applying conventional flexural theory together with these factors.

ii) Nonlinear Frame Analysis

Firstly, sectional analysis is carried out using the stress-strain relations of the constitutive materials for finely divided fiber elements of the beam section, taking into account both material and geometrical nonlinearities. Next, a complete structural system comprising beam elements and external tendon elements is analyzed by means of frame analysis using the results of sectional analysis. In this method, beam theory is adopted for the beam elements while external tendons are modeled by truss elements able to resist only axial forces. This method is explicit and is suitable for general use. A typical model for a beam member is illustrated in Fig.7.

Umezu[30],Tamaki[34],Kounoue[38],Aisawa[39],and Arai[40]et al. have analyzed the behavior to failure of simple beams and continuous beams reinforced with external tendons by the use of this method, and all reported that the analytical results agree well with measurements.

In Table 1 are listed the results of nonlinear frame analysis of ultimate flexural strength and deflection for the case of a simple T-shaped girder with a span of 30m and an overall depth of 2m as well as for a three-span continuous box girder with spans of 20-30-20m and an overall depth of 1.5m, assuming the ultimate compressive strain of concrete to be 3500μ. The ultimate strength and ductility expressed by the ultimate deflection are found to be markedly affected by the mix of external and internal tendons. In particular, it should be noted that the ductility of beams external tendons alone is markedly lower than when the mix is 50% or with internal tendons alone.

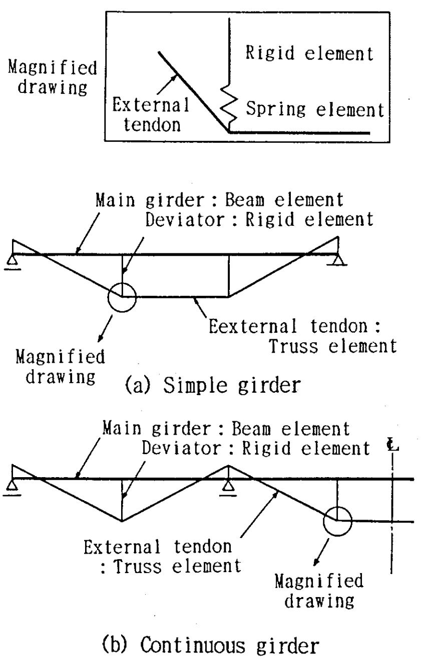


Fig.7 Typical example of model for nonlinear frame analysis[38]

Table 1 Analyzed results based on nonlinear frame analysis[38]

Cases	Forms of structure	Tendons		Ultimate state	
		Inner	External	Load (MN)	Deflection (m)
C-1	girder (cast-in-situ concrete)	100%	0%	4.42	1.44
C-2		50%	50%	3.76	1.19
C-3		25%	75%	3.42	0.91
C-4		0%	100%	2.89	0.23
C-5	continuous girder (cast-in-situ concrete)	100%	0%	2.80	0.62
C-6		50%	50%	2.63	0.59
C-7		25%	75%	2.52	0.50
C-8		0%	100%	2.19	0.19

iii) Nonlinear Finite Element Method
Externally prestressed beams have also been analyzed by the finite element method, taking into account both material and geometrical nonlinearities. Yaginuma[14] and Fujii at al.[41] report that flexural behavior can be estimated well by this method.

iv) Others
Virlogeux[42] proposed a nonlinear analytical method considering various factors inherent in external prestressing. In this method, the tensile force in external tendons is first regarded as an externally applied load, and the member deformation due to all applied loads is calculated taking into account constitutive material nonlinearies. Next, the change in tendon length, or the tensile force in the tendons, is estimated again, and iterative calculations are carried out until the tendon force converges to within a given error. The nonlinear behavior under increasing loads can be estimated in this manner.

Incidentally, the treatment of friction and slippage of external tendons at the deviators, which is important in the analysis of external prestressing, is described later in (3).

(2) Analytical methods for segmental structures

In the analysis of the segmental structures, the characteristic of the joints between segments are treated in the following ways.

i) Method considering local rotation at joint
The treatment of flexural rigidity after crack opening at joints is especially important in the analysis of externally prestressed segmental members.

Muller et al.[43] considered the bending moment(M)-rotation(θ) relation at crack opening joints for a given axial force(N), as shown in Fig.8, and calculated the deformation of each segment by means of the finite element method. Calculations were carried out assuming that the stress distribution at the joint contact zone is linear and the concrete between two adjacent joints is notcracked. The analytical result was quite consistent with measurements carried out on a tested simple beam, and it was confirmed that the proposed method was a satisfactory tool for predicting behavior up to the ultimate state of segmental girders, as shown in Fig.9.

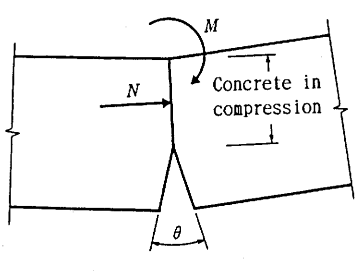


Fig.8 Local rotational deformation at joint[43]

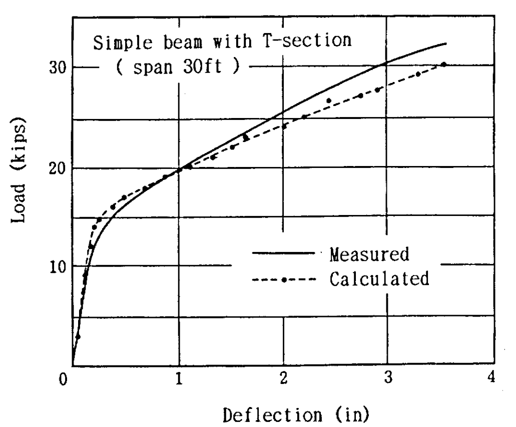


Fig.9 Comparison between measured and calculated load-deflection curves[43]

Virlogeux[42] proposed an analytical method that takes proper account of both local stresses and rotation occurring at a cracked joint, since rotational deformation is concentrated at a joint opened within a critical region in a segmental structure.

Fujii et al.[41] applied the nonlinear finite element method, assuming that the coefficient of friction at the joint surface is zero and that two adjacent upper flanges are tightly fixed at the joint to prevent structural instability. It was indicated that the flexural behavior of segmental beams with external prestressing could be simulated well by this method.

ii) Method considering discontinuity of structural properties

Arai et al.[40] analytically studied the difference in behavior between segmental and monolithic beams, using a simple-span beam and a three-span continuous beam with external tendons. In calculating the segmental beam based on nonlinear frame analysis, the regions provided with ordinary steel were modeled as reinforced concrete sections, while those without steel were treated as non-reinforced ones. It was indicated that flexural strength and deformation in the ultimate state of the segmental beam was less than that of the monolithic one, both for simple and continuous beams, and that an ultimate strength in excess of 90% of that of a monolithic beam could be attained when the ratio of internal tendons to all tendons was more than 25% for a simple beam and more than 50% for a continuous beam.

Nakamura et al.[33] carried out loading tests and nonlinear analysis on a ⅓-scale model simulating a two-span continuous girder with each span 47m in length. This bridge is a total of 1,900m long and is a viaduct comprising 45 girders in 9 groups. It is the first large-scale bridge in Japan to fully utilize precast segments and external tendons with two types of slab keys and web shear keys. The analysis was conducted by modeling the regions within 50cm of a segmental joint as non-reinforced concrete and others as reinforced concrete in a similar manner to Arai's method mentioned above. It was concluded that the calculated stress increase in external tendons and the ultimate strength are somewhat smaller than the measured values, but that the overall behavior can be predicted with satisfactory accuracy by this method from a viewpoint of practical design.

(3) Treatment of friction at deviators

Two very important factors affecting the behavior of externally prestressed members are friction and slippage of the external tendons at deviators.

In analysing their structural behavior, methods of treatment of friction at the deviators are as follows.

i) Neglecting friction at deviators

Arai et al.[40] proposed a model in which the truss elements(external tendons) are combined with beam elements(concrete girders) by means of inclined spring elements such that axial forces in the tendons are equal on both sides of a deviator(rigid element), assumimg no friction ever exists at deviators.

Nakamura et al.[33] carried out analysis based on both the sliding model without constraint against external tendons at deviators and the fixed model as shown in Fig.10, and concluded that the sliding model could be applied with sufficient accuracy.

ii) Considering friction at deviators

Virlogeux[42] proposed a nonlinear analytical method for structures with external tendons as described in 2.3 (1) iv), and considered the friction and slippage at deviators as follows.

Slippage of the tendon can be judged not to occur when the relation below is satisfied between final tensile forces in the tendon on both sides of each deviator obtained by the trial-and-error procedure.

$$F_i \cdot e^{-[\mu \Delta\alpha_i + \lambda \Delta\ell_i]} \leqq F_{i+1} \leqq F_i \cdot e^{[\mu \Delta\alpha_i + \lambda \Delta\ell_i]} \quad (1)$$

where,

F_i, F_{i+1} : tendon tensile forces in block [i],[i+1] on both sides of deviator

$\Delta\alpha_i$, μ : angular change and its coefficient of friction

$\Delta\ell_i$, λ : length of winding path at deviator and its coefficient of friction

When Eq.(1) is not satisfied at a given deviator, slippage of the external tendon occurs at that deviator. It is thus necessary to confirm that Eq.(1) is satisfied at all deviators by repeating the calculations considering the slippage. In Fig.11 are shown the analytical results obtained in examining the slippage effect in a simply supported beam with two symmetrical deviators.

In addition, Kawamura et al.[44] conducted an analysis in consideration of nonlinear slippage of external tendons at deviators together with material and geometrical nonlinearities, in which the nonlinearity of slippage means that the difference between tendon forces on the two sides at a given deviator is nonlinear. Then the amount of slippage was estimated considering that ① no slip occurs at a deviator when the above-mentioned difference in tendon forces can be fully supported by friction at the deviator and ② slip occurs in response to the force difference if it is beyond the frictional force. In the latter case, it is necessary to carry out the calculations repeatedly until no slip occurs at any deviators, since secondary effects are caused by the redistribution of tendon forces. This method is based on a concept similar to Virlogeux's mentioned above. Comparing the calculated tendon forces predicted by Kawamura's method with the measurements in Fig.12, it is clear that calculations taking into account the tendon friction at deviators give results that are very close to the measurements.

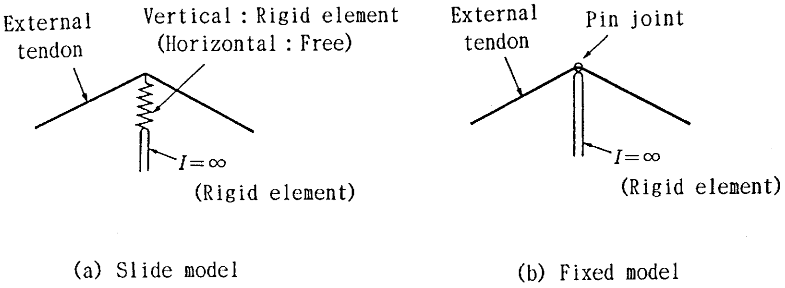


Fig. 10 Examples of analytical model for deviator[33]

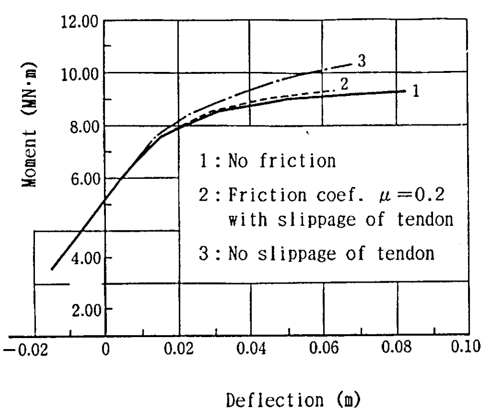


Fig. 11 Effect of tendon slippage at deviator on beam deflection[42]

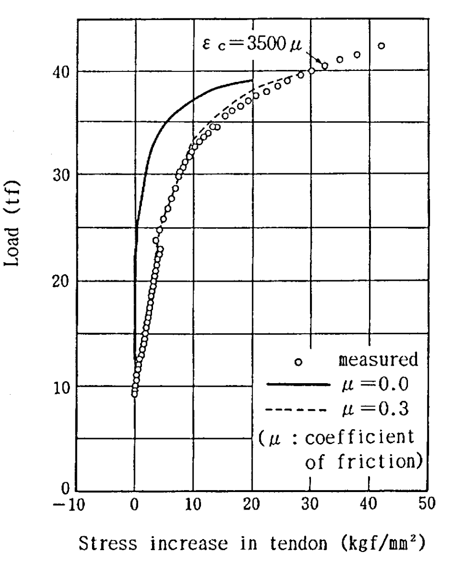


Fig. 12 Effect of friction at deviator on stress increase in external tendon[44]

3. ANCHORAGE STRENGTH AND LOCAL STRESSES AT DEVIATORS

A design must be properly drawn up such that the anchorage has sufficient strength to fully develop the tendon force and the deviators are able to resist the deviation force of the tendons.

3.1 Anchorage Strength

External tendons are usually anchored at partially widened diaphragms. Three-dimensional finite element analysis and model tests were carried out to examine the stresses in reinforcing steel after cracking and the final failure mode for anchorage of this type[45]. It was revealed that first the junction between diaphragm and web failed due to beam-like flexure and thereafter the reinforcing steel yielded. Finally, the lower flange failed abruptly under the combined action of flexure and punching shear. Although the anchorage had enough strength from a design aspect, its form was modified to improve resistance to this failure mode, as shown in Fig.13; here, the diaphgram for the anchorage is constructed monolithically with the upper flange.

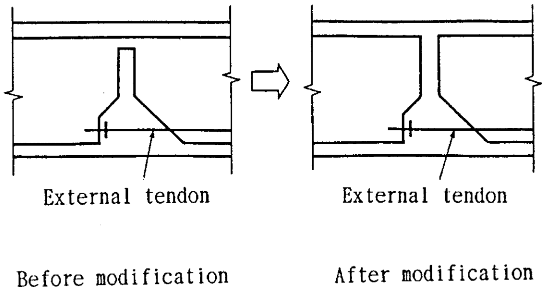


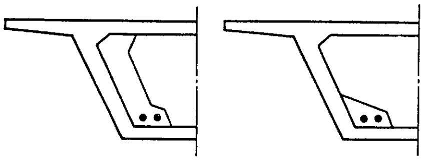
Fig.13 Shapes of anchorage for external tendon[45]

In applying external prestressing to the repair or strengthening of damaged structures, the method of anchoring tendons is of crucial importance. Model tests[41] to assess the shear transfer strength of concrete brackets installed on the surface of a girder web as anchorages for external tendons were carried out[41]. The specimens were produced as follows: 1) the surface of the girder web was roughened by chipping; 2) concrete for the bracket was cast in contact with the roughened web; and 3) prestressing bars in ducts through the bracket and girder web were tensioned to fasten them firmly. Test results showed that the measured shear transfer strength agreed relatively well with an analytical value obtained by setting all of the partial safety factors in the JSCE design equation based on the shear friction hypothesis to 1.0. The strength reduction due to tapering of the web could also be well assessed by this method. In other tests [45] where the inner surface of a box gider web was chipped to achieve an unevenness of about 25mm and the cast-in-situ concrete bracket was fastened with prestressing bars, the shear transfer strength was also estimated using the shear friction equation assuming a coefficient of friction of 1.0.

Numerous examples of the use of cast-in-situ concrete and steel brackets have been reported, but there are only a few instances of precast concrete brackets being used to simplify the field work. One reported example[47] describes how prepacked mortar made flud with a superplasticizer was filled into the aperture between the roughened main girder surface and the precast concrete bracket, with the two fastened by prestressing bars after hardening of the filled mortar. The shear transfer strength for this type bracket was also found to be predicted with adequate safety on the basis of the JSCE design equation.

3.2 Deviator Behavior and Design Details

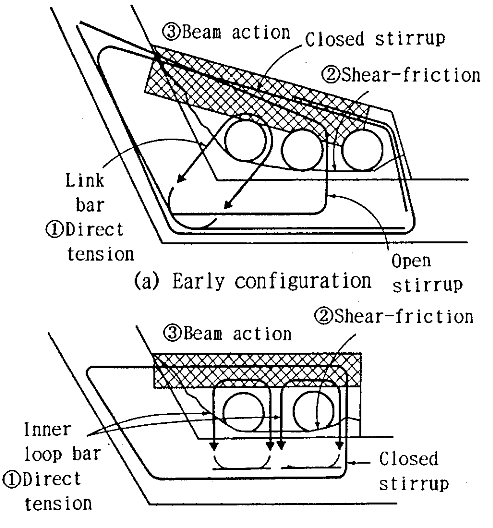
Three deviator types have been utilized in externally prestressed box-girder bridges, as shown in Fig.14. The advantage of (a) the diaphragm type and (b) the rib type is that compressive strut action may be utilized to resist tendon deviatory forces, so these designs have better resistance to deviator forces than (c) the saddle type. However, the dimensions of the deviators are large with the former two types. Further, the formwork for the diaphragm and rib and the geometry of the tendon pass-throughs are very complicated, especially for a curved span where the bridge curves while the tendons remain on straight paths. Therefore, use of the saddle type is likely to increase in externally prestressed bridges. With this type of deviator, however, tendon deviation forces cannot be resisted by the concrete compressive strut, so reinforcement is more congested than for a diaphragm or rib type.



(a) Diaphragm type

(b) Rib type (c) Saddle type

Fig. 14 Forms of deviator[48]



(a) Early configuration

(b) Modified configuration

Fig. 15 Reinforcement details for saddle-type deviator and simplified analytical models[48]

Beaupre et al.[48] carried out tests on the two saddle-type deviator specimens shown in Fig.15: 1) conventional reinforcement scheme consisting of primary link bars supplemented by two types of stirrups designated as open stirrups and closed stirrups, and 2) modified (simplified) reinforcement scheme consisting of small rectangular closed loops and outer closed stirrups. The test results were compared with analytical values based on the simplified model and the strut-tie model. The main results are as follows.

① Both the conventional reinforcement scheme and the modified scheme performed well under service loads and had a sufficient safety factor in the ultimate state.

② These two types of reinforcement scheme exhibited adequate ductility. The modified type, which resisted pull-out forces mainly by the direct tension of closed loops, had excellent ductility. In comparing direct tension reinforcement (link bar or loop bar) and top surface reiforcement, the former was significantly more effcient than the latter in resisting deviatory forces, and final failure occurred by failure of the direct tensioning steel.

③ From the cracking pattern and measured steel strain, three behavioral mechanisms became evident in the deviation saddle. These were the pull-out resistance of the direct tension reinforcement, the flexural beam-like resistance of the top surface reinforcement, and the shear friction resistance across the critical cracked plane.

④ The average ratio of experimental ultimate capacity of the ten specimens to the analytical values was 1.14 for the modified analysis model and 1.06 for the strut-tie analysis model.

In the modified reinforcement scheme, it is very important to provide loop bars around each individual tendon and to anchor their ends well. In addition, it is necessary to provide closed tie reinforcement to tie the deviator into the web and flange. The arrangement of closed sirrups in the same quantity and spacing as the loop bars should produce very satisfactory results.

On the other hand, Ito et al.[45] used the FEM method to analyze local stresses at deviators at the partially widened diaphragms located at mid-span and at the intermediate supports. The following results were obtained.

① Due to the angle change, the vertical component of prestress force acted on the deviator. As a result, the diaphragm behaved like a beam supported by the girder webs, and tensile stress in the transverse direction occurred in the upper portion of the diaphragm.
② A vertical tensile stress due to vertical component of prestressing force also occurred near the surface of the deviator.

Further, it was concluded that a slight increase in reinforcement at deviators would be sufficient from a viewpoint of practical design.

4. EXTERNAL PRESTRESSING FOR REPAIRING OR STRENGTHENING OF EXISTING STRUCTURES

External prestressing has been widely used for the rehabilitation of existing concrete bridges such as by repairing or strengthening them. It is suitable for this because of inherent advantages with respect to construction ease, cost, and maintenance. In References[49]～[61] a number of instances of rehabilitation are listed, and a number of typical examples are introduced here.

One example[41] is a case where crack opening under normal vehicular loading was detected at the joints in a post-tensioned segmental T-girder bridge on a Japanese highway. The bridge had been in service for about 30 years. Field loading tests using dump trucks indicated that the existing amount of effective prestressing force was about 30% less than the standard design value in the most severely damaged girder. The effect of applying external prestressing to rehabilitate this bridge was investigated by carrying out loading tests on damaged model beams and by nonlinear finite element analysis. Thereafter, the damaged bridge was repaired by means of external prestressing, and tests were carried out to examine directly the effect of the repair using dump trucks. It was confirmed that external prestressing was very effective as a method of repairing damaged concrete bridges.

External prestressing is being used not only to repair damaged concrete structures, but also to strengthen existing road bridges in Japan to cope with a newly established vehicle loading standard called the B-type live load, which is roughly 25% larger than the traditional one. The amount of external prestress needed to meet the B-type loading standard and its strengthening effect on an existing bridge were examined through loading tests using trucks. Theoretical analysis was also carried out[62]. Results indicated that the stress in external tendons due to applied loads and the loss in prestressing force due to creep and shrinkage of the concrete could not be assessed by grid analysis. A small part of the applied load was found to be carried by the external tendons according to the loading tests and three-dimensional frame analysis, but girder deflection and concrete stresses due to the applied load were scarcely affected. Grid analysis provided a practical and safe solution, since both creep and shrinkage of the concrete would be almost finished in most of existing structures. Although the theoretical values obtained by thin shell analysis, in which the main girders, diaphgrams, and slabs were treated as individual shell elements, were very close to the measurements, a more rigorous modelling including the tendons should be investigated, as this method overestimates the effects of external tendons.

In an example in Canada, a cast-in-place segmental box-girder bridge with a central span of 181m was experiencing distress due to increasing deflection accompanied by cracking. To remedy this situation, the external prestressing technique was adopted[46]. External tendons were anchored to webs inside the box girder by means of situ-cast concrete blocks as described earlier[41]. At the anchorage sections, each concrete block was linked by two diaphgrams to eliminate tensile stresses due to the induced bending moment in the box-girder webs. After external prestressing, the effectiveness of the repair was evaluated on the basis of measured deflections, etc., and the appropriateness of the design assumptions for strengthening was also examined[63].

A reinforced concrete bridge(11spans × 25m = 275m) built in Italy about 30 years ago suffered severe damage primarily as a result of inadequate reinforcement in the design, and flexural and shear cracks, steel corrosion, and cover spalling were in evidence. This damaged bridge was repaired by external prestressing, and the effectiveness of the repair was examined[64]. It was shown that the fundamental frequency increased significantly, and the ratio between measured frequency before and after the repairs was very close to the computed value obtained on the basis of sectional rigidity of the cracked and uncracked sections before and after the repair, respectively. This implied that measurements of fundamental frequency by means of dynamic vibration tests is very useful in the assessment of the rehabilitation of a large structural system.

The use of newly developed materials as external tendons has been attempted. In Japan, for instance, the effectiveness of repairs to damaged post-tensioned beams by external tendons of carbon fiber reinforced plastics(CFRP) and aramid(AFRP) has been studied[65]. The damaged beams were modelled by severing some internal tendons to simulate severe damage, such as tendon rupture due to corrosion. In other countries, external prestressing using these new materials has been used to repair or strengthen damaged structures[66].

In Japan, an existing simply supported girder bridge is being converted into a continuous one by means of external prestressing with the purpose of accomodating heavier vehicles (B-type live loading), while also reducing traffic noise(by eliminating joints between adjacent simple girders), eliminating troublesome problem in maintenance(caused by the expansion and contraction joints), and improving earthquake resistance. Field loading tests and theoretical analysis indicate clearly that complete continuity as regards applied live loads could be attained by this method[67]~[69].

On the other hand, the shear strengthening of prestressed concrete composite beams using externally prestressed stirrups has also been studied[70]. Test results indicate that these stirrups are very effective in strengthening beams with inadequate shear strength, and both the diagonal and horizontal shear strengths were markedly increased by means of transverse prestressing. Transverse prestressing proved able to change a susceptibility to brittle shear failure into a more ductile flexural failure. As a result of this prestressing work, flexural strength was developed with a strand embedment length as short as about 60% of the recommended design values in the current AASHTO/ACI Code.

5. DESIGN METHODS FOR EXTERNALLY PRESTRESSED CONCRETE STRUCTURES

Various factors influence the structural properties of externally prestressed concrete, and there still remain several problems to be clarified. The important matters in the present representative design codes related to external prestressing are summarized as follows.

5.1 Calculation of Prestressing Force

Introduced prestress force can be estimated using Eq.(2) considering the loss due to friction at deviators. According to, for instance, PE/SETRA in France, the coefficients of friction in Eq.(2) are those given below.

$$\sigma = \sigma_o \cdot e^{-\mu\alpha - \lambda x} \qquad (2)$$

where,

Metal ducts : μ =0.2~0.3, λ =0

HDPE ducts : μ =0.12~0.15, λ =0

Greased strands : μ =0.05, $\lambda = 10^{-3}$/m

5.2 Degign of Anchorages

There are few instances of special provisions for the design of anchorages for external tendons. In external prestressing, however, a proper distribution of local stresses concentrated at the anchorages is essential.

The design method for conventional protruding anchorages as used with internal tendons might be applied to external tendon anchorages. When anchoring external tendons to protrusions installed on the girder webs, reinforcement arrangements and dimensions should be determined such that the large shear forces induced at the intersection between them are resisted.

5.3 Design of Deviators

It is specified in the PE/SETRA Standards that 1)deviators should be installed such that there are no sharp deviations in the external tendons, 2)the radius of bends in tendons should be at least 2~4 m, and 3)an aperture of more than 10mm should be provided between inner and outer ducts in the case of double ducts, considering an error in alignment of 5/100 rad between the two ends.

5.4 Resonance and Fatigue of External Tendons

(1) Resonance

When the natural frequency of the external tendon coincides with that of the girder, resonance may be induced by the impact of vehicles[71], resulting in fatigue failure due to repetitive flexural stress in the tendon near the anchorages.

Regarding design provisions aimed at preventing such vibrations of external tendons, the AASHTO Code, for instance, prescribes that the distance between two adjacent fixed points of the tendon should be less than 7.5m(25ft) if no detailed vibration analysis has been conducted. Similarly, the RE/SETRA Standards provide that the external tendon should be supported at a spacing of 10~15m so as to prevent a coincidence of natural frequencies between tendon and girder.

According to vibration tests[57], the natural frequency of an external tendon is influenced by its tensioning force, and its vibrational behavior can be predicted from that of the main girder if the structural properties of the main girder and tendon are known. In practice, it is very important to accurately predict the damping of vibrations transferred into the tendon from the main girder.

(2) Fatigue

There is a practical need to examine the effect of external tendon bending radius at deviators on fatigue strength.

Ito et al.[45] carried out fatigue tests on external tendons to validate a bend radius of 3.5m at a deviator. This is quite a small radius compared to that of a convenional internal tendon (for instance, 8.0m is given by the Japan Specifications for Highway Bridges). This value was chosen considering the typical design bending radius of external tendons prescribed in AASHTO (3.0m) and SETRA (2.5m). After 2×10^6 cycles of tensile loading, neither the ordinary uncoated prestressing strands nor the epoxy coated strands suffered any damage such as failure of component wires.

Further, the fatigue strength of the anchorages also needs to be examined in the case of unbonded tendons, as noted in many specifications including the JSCE Standards. For instance, CEB-FIP 90 prescribes the design fatigue strengths of tendons and anchorages, and DIN 4227 provides limit values for the stress amplitude in tendons under repeated loading.

5.5 Ulitimate strengths

In calculating the ultimate flexural strength of prestressed concrete members with unbonded tendons, as used typically in external prestressing, two distinct methods are adopted in practical design. One is the method prescribed in CEB-FIP 90 and the Japan Design Standards for Railway Structures, etc., in which the effective prestress force in an unbonded tendon is treated as an external load and is not included in the calculation of resisting moment. Another method is to estimate the ultimate flexural moment based on the ultimate tensile stress in an unbonded tendon as provided in BS 5400, ACI 318-95, and DIN 4227, etc. Incidentally, the JSCE Standards specify that the ultimate strength of flexural members with unbonded tendons should be reduced to 70% of that of bonded ones when a rigorous analysis is not conducted.

A comparison of the calculated ultimate flexural strengths of unbonded members based on several representative design equations with numerous measured results has been made to examine the accuracy of each equation[72] that is useful in practical design.

In examining the safety of externally prestressed structures at the ultimate limit state, estimating their ultimate flexural strength by neglecting any stress increase in the external tendons provides a safety margin. However, this procedure results in an uneconomical design, since the total number of tendons increases if the ratio of external tendons to all tendons is relatively high. In such cases, it may be reasonable to assess structural safety based on the overall structural system by an appropriate nonlinear analysis, and such an examination has also been reported[34].

On the other hand, the design shear strength of externally prestressed beam members can be predicted with safety by applying the JSCE Standards. However, the effect of external prestressing (shear strength carried by concrete and shear force shared by its vertical component) has to be studied in still more detail[20],[24],[25].

6. CONCLUSIONS

Research on externally prestressed structures aimed at clarifying their behavior and developing analytical methods has been very active. However, to establish a rational design method for such structures, several problems still need to be clarified further.

(1) Resonance of external tendons with the structure
(2) Rational and practical stress distribution methods and reinforcement details at anchorages and deviators
(3) Proper sefety evaluation for fatigue failure of external tendons with bending at deviators
(4) Precise design equations for estimating ultimate flexural and shear strength of externally prestressed members including continuous beams
(5) Reliable method of protecting external tendons from corrosion over a long period

Considering the several benefits inherent in external prestressing, both as regards structural characteristics and constructional aspects, this prestressing technique could find application in a wider range of fields. Further extensive development might be expected in the future.

I would like to express my sincere appreciation to the many authors whose names are given in the references for allowing me to quote their extensive and valuable literature.

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