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Study of a New Structural Type for Prestressed Concrete Bridges

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Summary

In order to achieve a more efficient external tendon layout in prestressed concrete continuous girder bridges, a non-linear analysis was made of a new layout with tendons extending above the girder over the supports as in an extradosed bridge, and below the girder in the span as in a beam string structure. A model group was studied in which the tendon quantity and eccentricity were varied gradually under a constant service load.
The large eccentricity tendon layout allowed the quantity of prestressing tendons to be reduced by 35% as compared to conventional layouts, while maintaining the required flexural capacitys.
Structures with large eccentricity tendon layouts were found to have the following advantages:
Economical, and Greater flexural capacity, as advantage is taken of the tension increase in the tendons, and Greater freedom of tendon layout, depending on the required performance.

1. Introduction

Features of extradosed concrete bridges and other large-eccentricity external-tendon prestressed concrete bridges include the ability to use lower girders owing to the large eccentricity of the tendons compared with girder bridges, the ability as a result to reduce the girder weight, and lower stress amplitude due to the live load compared with cable-stay bridges, so that designs can assume higher stress-limit values taking fatigue into account than in designs of cable-stayed bridges. And, the fact that the tower of an extradosed concrete bridge is not very high makes this an attractive design when scenery and environmental conditions are considerations.

Although the advantages of large-eccentricity external-tendon designs are thus becoming clear, numerous aspects of the structure of such designs remain unclear. The authors previously conducted loading experiments using indoor models and nonlinear analyses in an attempt to determine the basic characteristics of large-eccentricity external-tendon prestressed concrete continuous beams, and confirmed that such structures possess sufficient flexural capacity and offer superior economy[1]. In the present study, we performed parameter analyses in order to determine the effects of the prestressing tendon layout and eccentricity on the girder performance for the service load and ultimate load, using a prestressed concrete bridge model with actual-scale spans. Below we report our results.

2. Analysis Model

Existing extradosed concrete bridges employ eccentric towers at central supports, and by increasing the eccentricity of tendons can efficiently deal with the negative flexural moment occurring in the girders. On the other hand, the beam string structure with large tendon eccentricity and with struts placed beneath spans is used to efficiently deal with positive flexural moments. Such large-eccentricity external tendons above supports and below spans serve a similar purpose in that they effectively cancel girder flexural moments. The model studied in this work appears in Fig.1; tendons are placed above and outside girders at supports and in spans, incorporating the structures of both extradosed bridges and beam string bridges in the large-eccentricity tendon model. The model studied was an equal-girder-height box-girder bridge with three continuous spans of 75 m each. Materials used in components are indicated in Table 1. The model design conditions are described below.

(1) All prestressing tendons in the longitudinal direction was present in the form of external tendons. The effective tensile stress of tendons was taken to be 50% of the tensile strength.

(2) From 0.17 to 0.25% of the cross-sectional area of the girder was reinforcing steel, so as to satisfy the condition of a minimum 0.15% steel in the girder.

(3) In the "1.0 Ap model" the tendon layout is internal, as in the case of internal tendons. However, in analysis the tendons were treated as unbonded tendons, as in the case of external tendons.

(4) The names of the "0.76 Ap" and similar models indicate the Ap ratio with the conventional (1.0 Ap) model of the total cross-sectional area Ap of external tendons in the cross-section at the central support.

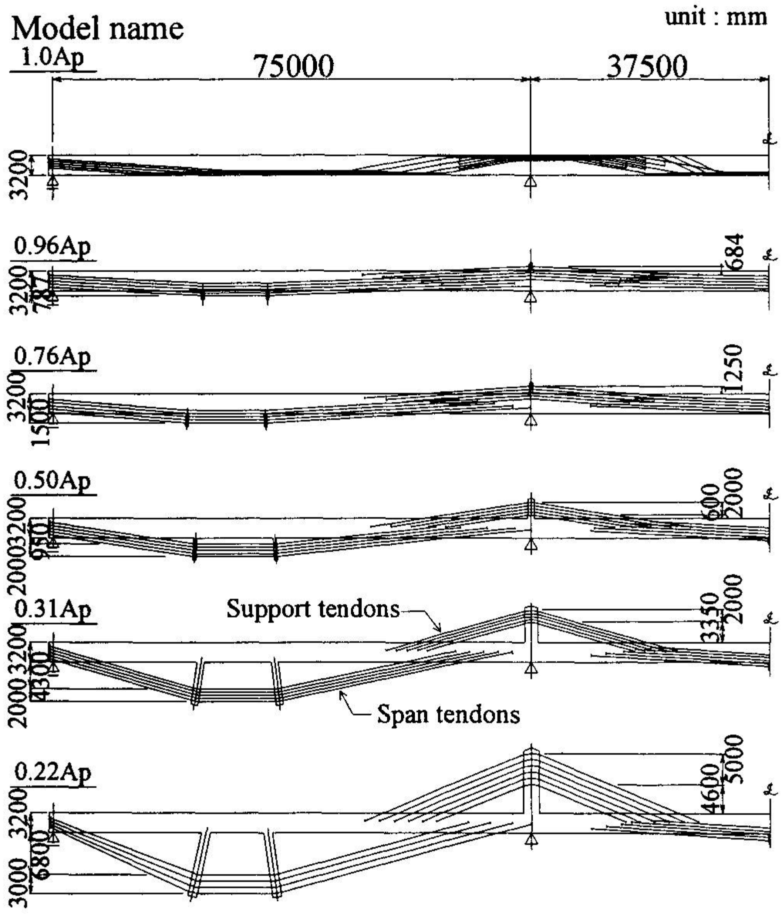
In the 0.22 to 0.96 Ap models, the tendons have a concordant layout.

(5) Struts positioned underneath spans are positioned in a direction so as to bisect the inner angle of the deviator. This was done in order to use struts supporting the tendon tension component as axial

Table 1. Materials used

(N/mm^2)

Concrete	Design criterion strength	40
PC Strands 12S15.2 SWPR7B	Tensile strength	1860
Reinforcing steel bar SD345	Tensile strength	345



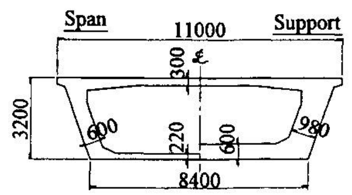


Fig.1 Structure of models studied

force-predominant materials. In the large-eccentricity model, external tendons were fixed to girders at both ends of cable arrangement intervals, and deviators of towers and deviators of struts, external tendons were kept moveable.

(6) The tendon quantity in these models was laid out so as to satisfy requirements for the allowable stress at the concrete girders for in-service load. The tendon eccentricity in the major cross-section is set such that stress by prestressing forces at the concrete girder's tensioned edge is equal over the entire model. The tensioned edge combined stress at the in-service load appears in Table 2. The relationships between the external tendon total cross-sectional area at the main cross-section and the eccentricity is plotted in Fig. 2.

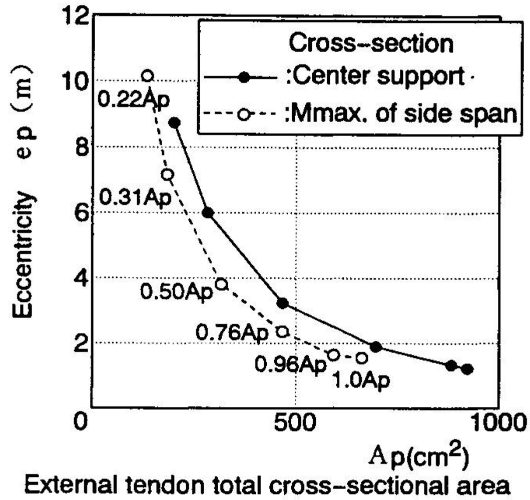


Fig.2 Relationships between total external tendon cross-sectional area and eccentricity in major cross-sections for each model

3. Design by Conventional Method

3.1 Stress amplitudes for in-service loading

Table 3 shows stress amplitudes in tendons occurring in B live loading as determined from linear analysis. The smaller the tendon layout quantity of the model, the greater are the stress amplitudes. Moreover, we see that in all models amplitudes are greater for span tendons than for support tendons. We also note that for the Odawara Blue Way Bridge and Tukuhara Bridge built by Japan Highway Public Corporation, both extradosed concrete bridges the in-service load allowable stress of the tendon tension for which was set at 60% of the tensile strength, the stress amplitudes due to the live load is 37 N/mm ^{2} [2]. It is thought that tendons for which stress amplitudes under a live load are large must be designed with consideration paid to safety with respect to fatigue, including fretting at the deviator.

Table 2. Combined flexural stress of girder cross-section for in-service loading

(N/mm²)

	Mmax of side span		Central support	
Model	Upper edge	Lower edge	Upper	Lower
1.0Ap	12.0	−1.1	0.4	11.5
0.96Ap	11.2	−1.0	0.1	11.7
0.76Ap	9.0	−1.0	0.0	9.3
0.50Ap	6.3	−0.9	0.0	6.3
0.31Ap	4.1	−1.4	0.0	4.0
0.22Ap	3.6	−1.4	0.0	3.5

allowable stress span : $-1.5 < \sigma < 14$N/mm ^{2}
support : $0 < \sigma < 14$N/mm ^{2}

Table 3. Stress amplitudes in external tendons due to live loads

(N/mm²)

Model	Support tendon	Span tendon
1.0Ap	7	20
0.96Ap	5	17
0.76Ap	6	21
0.50Ap	11	28
0.31Ap	14	43
0.22Ap	20	54

3.2 Study of conventional designs in ultimate loading

Table 4 shows the flexural failure safety factor under ultimate loading for the conventional model (1.0 Ap). When tendons are regarded as internal bonded tendons in calculations, the safety factor is 1.25 to 1.33; but calculations as external tendons without anticipating an increase in tensile stress an insufficient safety factor of 0.71 to 0.80.

Table 4. Flexural failure safety in ultimate loading

($\times 10^3$kN·m)

Cross-section	Acting moment Md *1	Case assumed to bonded tendon		Case assumed to unbonded tendon *2	
		Resistance Moment Mu	Safety Mu/Md	Resistance Mu	Safety Mu/Md
Mmax of side span	245	318	1.30	175	0.71
Central support	289	385	1.33	229	0.79

*1) Md = 1.7 × (Dead loads + Live loads + Impact) + (the indeterminate forces due to prestressing forces)
*2) calculations as external tendons without anticipating an increase in tensile stress due to deformation of stuctural members

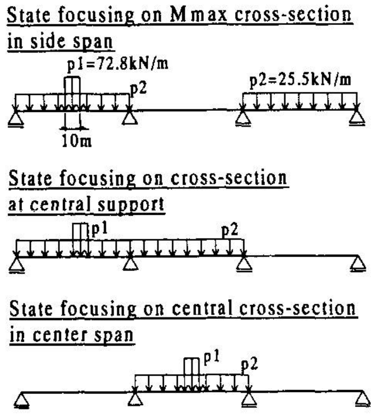


Fig.3 Loading state of Live loads and impact

4. Nonlinear Analysis

4.1 Analysis conditions

In analysis for ultimate loading, the "Say-NAP" fiber model program[3] was used, taking the material and geometrical nonlinearity into account. Stress-strain curves for the materials used conformed to the Standard Specification for Design and Construction of Concrete Structures of the Japan Society of Civil Engineers. The concrete tensile strength was taken to be zero.

In analysis, the dead load D and the B live load L in the loaded state (Fig. 3), as well as the impact I, were increased gradually, and the course from in-service loading until material ultimate loading was traced. In the case where the load was gradually increased, the loading shown below was adopted, taking into account the load combination under ultimate loading indicated in the Concrete Bridge Section of Specification for Construction of Road Bridge .

Gradual increase in load,

case : $\gamma \times (D + L + I)$

Where, γ is the load increment coefficient ($\geqq 1.0$).

The load was thus increased gradually, and at all cross-sections of the girder, the point at which the concrete compression edge reached ultimate strain (=0.0035) was taken to be flexural failure, and calculations were halted.

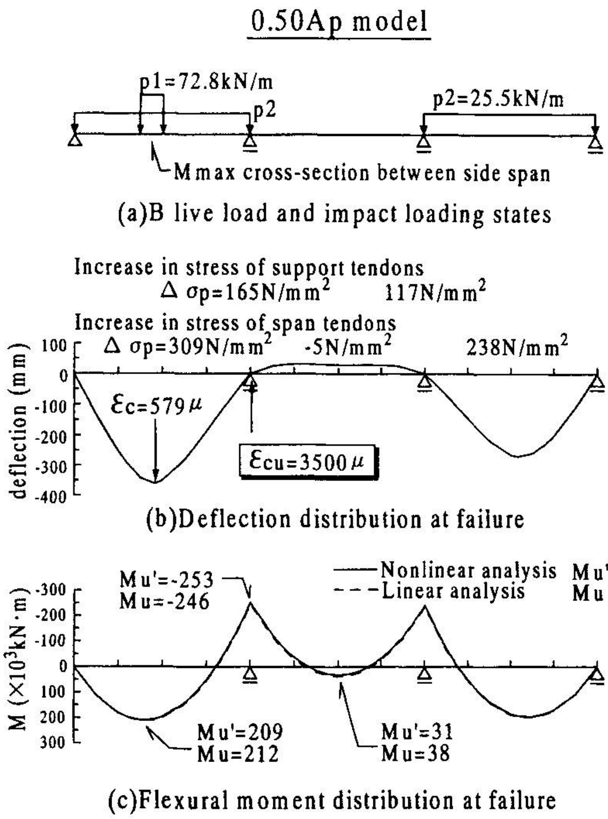


Fig.4 Girder deflection and flexural moment distribution at failure

4.2 Analysis results

(1) Girder deflection and flexural capacity

Figure 4 shows the girder deflection and flexural moment distributions at failure

for the case of live loading, focusing on the Mmax cross-section in side span for the 0.50Ap model. In all loading cases for all models, the failed cross-section was the central support cross-section. This was inferred to occur because the increase in tension is smaller for the support tendons than for the span tendons, so that the flexural capacity of the support cross-section was not increased. In these models, there is not much moment redistribution, and so there was little difference between the flexural moments in the linear and nonlinear analysis.
Figure 5 indicates the relationships between load increment coefficient and girder deflection for each model for the case focusing on the Mmax cross-section in side span; Figure 6 shows the flexural moment (flexural capacity) occurring in the cross-section in question when the center support cross-section undergoes flexural failure in each case. Compared with the conventional model (1.0Ap), nearly all the large-eccentricity models had about the same coefficient γ and flexural capacity at failure and the same deformability as well.

For the present model, which is an entirely external-tendon model, in the case of gradual increase with $\gamma \times$ (D+L+I) flexural failure occurs before reaching γ =1.7; this is because the tendon layout quantity is determined by the required quantity at service load. Analysis to satisfy the requirement for γ is discussed in "5. Study of Tendon Quantity Satisfying Safety Requirements at Ultimate Loading" below.

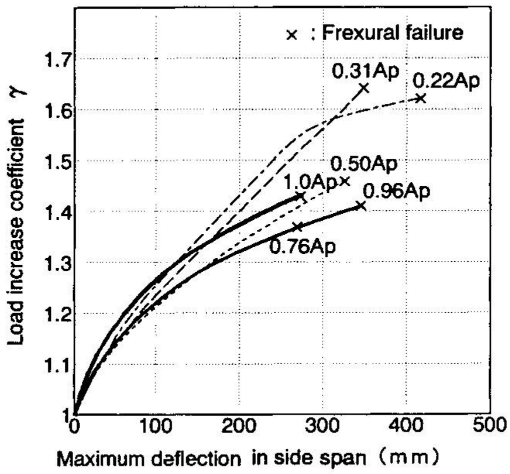


Fig.5 Relationships between load increase coefficient and girder deflection

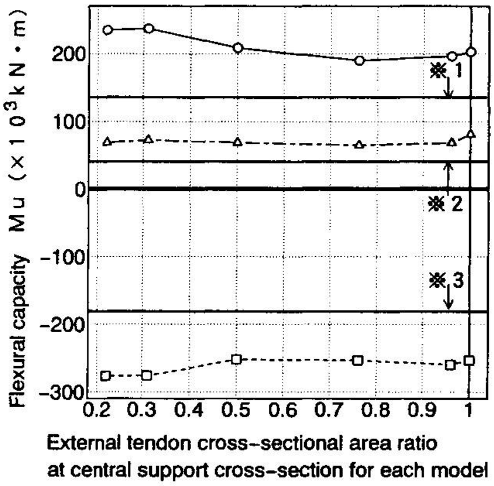


Fig.6 Frexural capacity for each model

※ 1 : Side span Mmax cross-section for 1.0 × (D+L+I)
※ 2 : Central cross-section in center span for 1.0 × (D+L+I)
※ 3 : Center support cross section for 1.0 × (D+L+I)
—o— Loading state focusing on Mmax cross-section in side span
--△-- Loading state focusing on central cross-section in center span
--□-- Loading state focusing on cross-section at central support

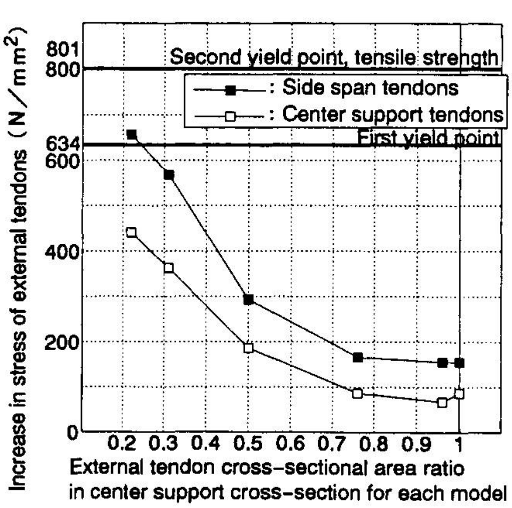


Fig.7 Increase in stress at flexural failure for each model

Table 5. Rate of increase of external tendon layout quantity to satisfy safety requirements in final loading

Model	Original tendon layout	Case where tendon layout quantity is increased	
	Loading coefficient γ at failure	Rate of increase of layout quantity	γ
1.0Ap	1.42	×1.25	1.71
0.96Ap	1.39	×1.27	1.71
0.76Ap	1.34	×1.35	1.76
0.50Ap	1.40	×1.30	1.78
0.31Ap	1.54	×1.15	1.74
0.22Ap	1.54	×1.20	1.75

(2) Stress increase in external tendons
Figure 7 shows the increase in stress of the external tendons in each model, from the effective tensile stress point up to flexural failure of the girder. From these results we see that the smaller the tendon layout quantity, the greater is the increase in tendon stress. The stress is from 70 to 440 N/mm^2 in the center support tendons passing through the failed cross-section. In the 0.22Ap model, a stress increase extending to the yield point was observed.

5. Study of Tendon Quantity Satisfying Safety Requirements at Ultimate Loading

In the present bridge model, as explained above, safety requirements to prevent flexural failure for load combinations at ultimate loading are not satisfied. In order to satisfy safety requirements for both in-service loading and ultimate loading in this entirely external-tendon model, it is necessary to focus on ultimate loading when setting the tendon layout quantity. In order to improve the girder flexural capacity, the following two approaches are conceivable. (1) Abandon an entirely-external tendon design in favor of a combined internal-external tendon design, with tendons in the longitudinal direction. (2) Increase the external tendon layout quantity. Here we adopted (2) and left the eccentricity of the external tendon group unchanged. Through a nonlinear analysis it was confirmed that the loading coefficient at failure γ is the required value (=1.7); results appear in Table 5. As a result it was found that for each model, the tendon quantity must be increased by 15 to 35%. This relative increase does not vary greatly from one model to another, so that from these study results as well we see that the large-eccentricity model has performance comparable to that of conventional models.
For the example of the 0.50Ap model indicated by shading in Table 5, in order to satisfy safety requirements for service and ultimate loads, the layout tendon quantity must be 0.65Ap (=0.50Ap × 1.30); but compared with conventional 1.0Ap internal tendon layouts the tendon quantity is decreased by 35%, and can be described as an economical design.

6. Conclusions

The results of this study may be summarized as follows.
(1) The smaller the external tendon layout quantity of a model, it was found, the greater the increase in tension with loading tended to be. For side span tendons in the 0.22Ap model, the yield point was reached at the time of flexural failure of the girder.
(2) Designs in which external tendons layouts have large eccentricities at supports and spans result in increased tension in external tendons due to deformation of structutral members, and so are regarded as an economical bridge design in which the tendon quantity can be reduced while maintaining the required flexural capacity.
(3) Large-eccentricity external tendon designs can be described as structures which afford a high degree of design flexibility and enable freedom in tendon eccentricity and quantity, with due consideration paid to stress amplitudes due to tendon live loads, stress and strain in ultimate loading, and other parameters.

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