Mechanical Behavior and Design Concept of Prestressed Composite Plate Girders with External Tendons

Ayaho MIYAMOTO*, Katsuji TEI**, Masumi GOTOU***

(Received July 11, 1995)

Abstract

The application of the prestressing technique to steel structures makes it possible to increase the load-carrying capacity as well as to extend the elastic region of existing steel bridge systems. The aim of this particular research is not only to develop a method of nonlinear analysis of prestressed composite plate girders based on the incremental deformation method, but also to demonstrate the feasibility of applying the prestressing technique to the strengthening of existing steel bridges. The accuracy of the proposed analysis was verified by comparisons with test results on full scale prestressed composite plate girders with various levels of prestressing, eccentricity of draped tendons, and differing tendon material properties. Using this method it was possible to predict the ultimate response and failure modes of the various prestressed composite plate girders. The performance of different types of prestressed composite plate girders under static loading was analytically evaluated for the strengthening of existing steel bridge systems. Furthermore, a method of performance evaluation related to the load-carrying capacity and deflection control of the prestressed composite plate girders with external tendons is proposed, based on consideration of the analytical results, with some optimum geometric and material parameters. Finally, the design concept of prestressed composite plate girders with external tendons is discussed in relation to the construction of new bridges, and in the strengthening of existing bridges.

Keywords : Prestressed composite plate girder, external tendon, mechanical behavior, bridge strengthening, design concept.

1. INTRODUCTION

Unlike concrete, steel exhibits almost equal strength in tension as it does in compression. So, from consideration of the material characteristics alone, there would appear to be no immediate advantage in introducing prestress in steel structural members. However, if the prestress can be used to reduce the stresses induced by the action of external live loads and structural dead weight, various applications can be considered for the use of prestressed composite steel structures. The usefulness being mainly in

*Professor, Dr. Eng., Dept. of Computer and Systems Engineering, Yamagutchi University.
**Graduate Student, M. Eng., Graduate School of Science and Technology, Kobe University.
***Technical Official, B. Science, Dept. of Computer and Systems Engineering, Yamagutchi University.

the strengthening of existing structures, and in the greater freedom of choice offered in the selection of materials and accompanying cost effectiveness, for the design and construction of new bridges. In this way, prestressed composite steel structures can be seen to offer various advantages over other steel structures[1][2].

The maintenance, repair, rehabilitation and strengthening of bridges in use around the globe have been receiving more and more attention in recent years, and the deterioration of existing bridges due to increasing overloading and progressive structural aging has become a major problem. The number of heavy vehicles and volume of traffic using these bridges has come to exceed the values predicted at the time of bridge design, with the result that many of these bridges are suffering from fatigue damage and are now in urgent need of strengthening. In addition, problems can also be expected in providing suitable countermeasures for cases where revisions in the design live load exceed the present load capacity of the existing bridge. Prestressing can be considered as one effective method applicable in such cases. At the same time, materials development is being actively pursued in such areas as high strength steels and continuous fiber reinforced plastics (FRP) that have high strength, outstanding corrosion resistance and high elasticity. If these materials can be successfully incorporated in prestressed steel structures, not only the strengthening of existing bridges, but the development of new bridge design configurations becomes possible.

This paper describes various studies on prestressed composite steel structures, carried out mainly with respect to the strengthening of existing bridges.

In prestressing composite structures, one method is to make use of the flexural restoring force of the steel member itself, and another is to use cables. The first method involves "preflexing" or re-distribution of bending moments within the member, while the latter generally involves controlling the internal stresses within the member by the application of forces through externally attached cables[1]. In the case of external cables being used for bridges, the Büchenauer Bridge in Germany and other European and American bridges have applied this method[3],[4],[5], but in most cases the prestressing was applied in the original design. In Japan, except for several bridges that were constructed at the end of the 1950s using prestressed composite plate girders [6], very little research has been carried out. The reasons for this can be attributed to such factors as the increased complexity of detail design of such parts as anchorages when prestressing is introduced, increases in the number and type of construction processes involved, corrosion protection of the prestressing tendons, and the fact that the benefits of introducing prestressing force are not as evident as they are in the case of concrete. However, in recent years, there have been significant advances in design and construction methods, and new materials have been developed. This coupled with the increase in the number of bridges requiring rehabilitation or strengthening, means there is more than sufficient reason to reconsider the possible advantages that prestressed composite structures have to offer.

In order to investigate the fundamental mechanical behavior of prestressed composite structures, a simply supported prestressed composite plate girder was made the subject of static behavioral analysis, taking such things as the cable arrangement and cable material as parameters. In addition full scale tests were carried out on prestressed composite plate girders to confirm the results of the analysis. Also, with a view to evaluating methods of strengthening existing bridges, the fundamental characteristics of prestressed composite plate girders (hereafter PS composite girders), namely the yield load, ultimate load, and flexural rigidity, were evaluated. A method to improve load carrying capacity by introducing prestressing force is also proposed, based on the results of the various analyses. Also, as a concrete example, the results of applying the method to an actual bridge were studied to evaluate the performance. The elasto-plastic analysis method and the verification methods used are described in detail in the authors' paper reference [6], an outline of which is given below.

2. ELASTO-PLASTIC ANALYSIS OF PRESTRESSED COMPOSITE GIRDER[8]

2.1 Fundamental Assumptions and Flow of the Analysis

Below is a description of the formulation of the incremental deformation method, one means of carrying out the elasto-plastic analysis that can be applied in the case of the PS composite girder for various cable materials and arrangements. The flowchart of the incremental deformation method is shown in Fig. 1[8]. The fundamental assumptions of this method are:

i) plane surfaces are maintained,

ii) the small deflection theory is applicable,

iii) there are no residual stresses in the girder,

iv) buckling of the girder will not occur before failure,

v) deformation due to shear forces is negligible,

vi) the cable strain is found assuming the extension of the cable and girder to be the same at the cable position,

vii) the non-linear shape of the concrete is assumed to be parabolic[7].

For the sake of convenience, it is assumed that failure initiates at the lower flange of the girder, and that the upper surface of the concrete deck slab achieves ultimate strain on failure. In actual fact, the strain at each position of the girder cross section is checked for each strain incremental step, and the yield or failure are judged.

2.2 Formulation of the Incremental Deformation Method

1) Strain at Each Location of the Composite Girder Cross Section

The stress-strain relation for the PS composite girder central cross section are shown in Fig. 2. For the incremental deformation method, instead of an applied load, the strain at the bottom of the lower flange is increased by a fixed increment ε_{s1}, and the strain at the various positions are given by the following equations(for symbols, see Fig. 2):

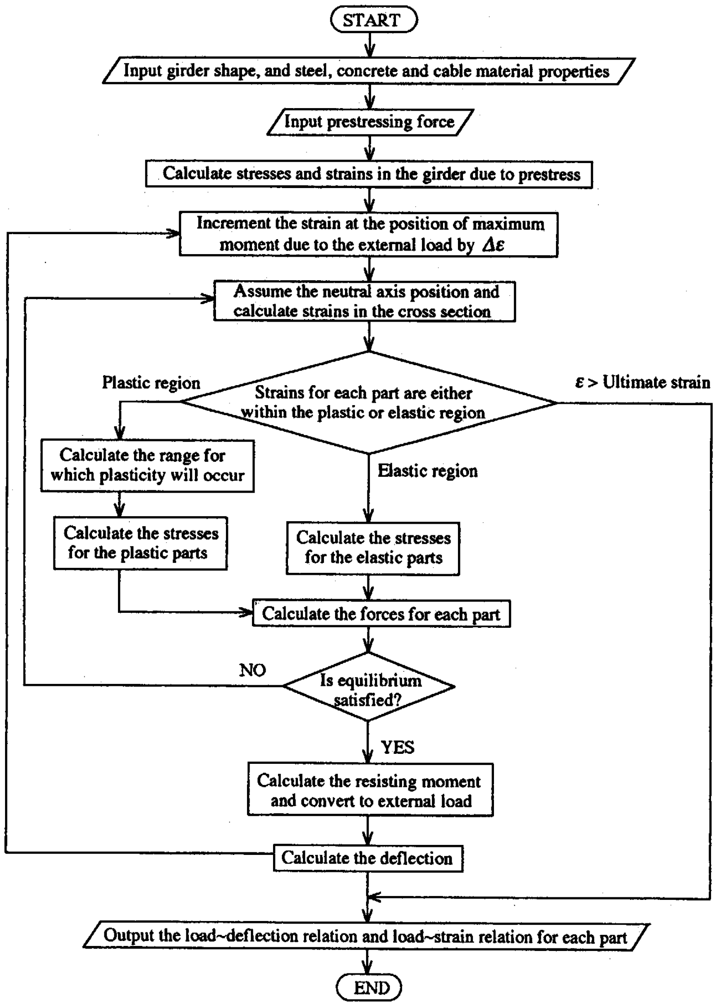


Fig. 1 Flowchart of Incremental Deformation Method

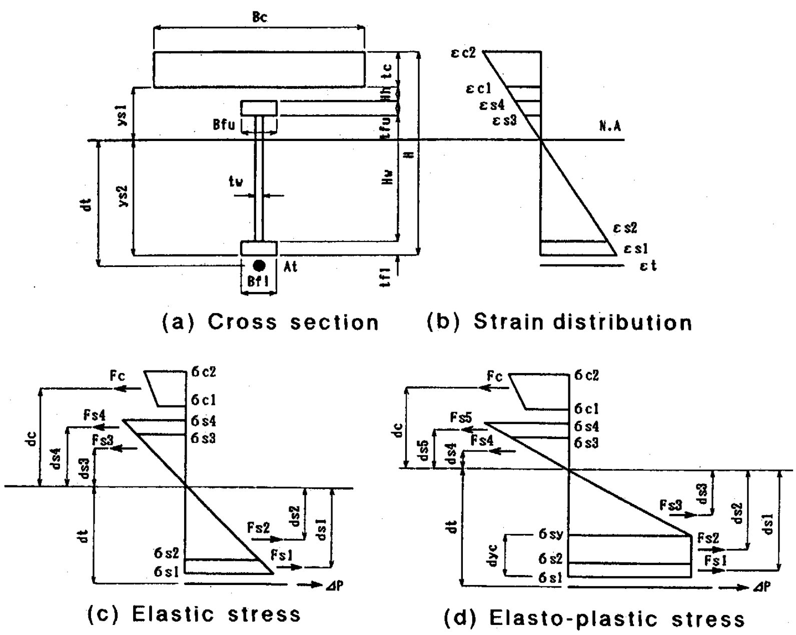


Fig. 2 Strain Distribution and Stress Condition in the Prestressed Composite Girder Cross Section

$$\varepsilon_{S2}=\frac{y_{S1}-t_{fl}}{y_{S1}}\varepsilon_{S1},\ \varepsilon_{S3}=\frac{y_{S2}-t_{fu}}{y_{S1}}\varepsilon_{S1},$$
$$\varepsilon_{S4}=\frac{y_{S2}}{y_{S1}}\varepsilon_{S1},\ \varepsilon_{C1}=\frac{y_{S2}+H_h}{y_{S1}}\varepsilon_{S1},$$
$$\varepsilon_{C2}=\frac{y_{S2}+H_h+t_c}{y_{S1}}\varepsilon_{S1} \tag{1}$$

2) Cable Strain

The cable is fixed to the steel girder and the girder end attachment, with the queen post arrangement[8], the cable is in contact with the saddle at the cable bend, but there is assumed to be no friction between the cable and saddle, the deformation due to shear force of the cable and composite girder is considered negligible, and the cable strain ε_t is considered uniformly distributed throughout the entire girder. Then the total girder deformation δ_s at the cable position given by Eq. (2) and the total cable deformation δ_t must be equal to satisfy the compatibility condition, so the cable strain can be determined.

$$\delta_s=\delta_t \tag{2}$$

If the cable length is l, then the total cable deformation δ_t will be given by the following equation.

$$\delta_t=l\varepsilon_t \tag{3}$$

In the case of a straight line arrangement as shown in Fig. 3, the girder deformation at the cable position integrated along the full length of the girder is the cable position deformation δ_s and is given by the following equations.

$$\delta_s=2\int_0^{a'}\frac{x}{a'}(\varepsilon_S+\varepsilon_{SH})dx+b'(\varepsilon_S+\varepsilon_{SH})-L\varepsilon_{SH} \tag{4}$$

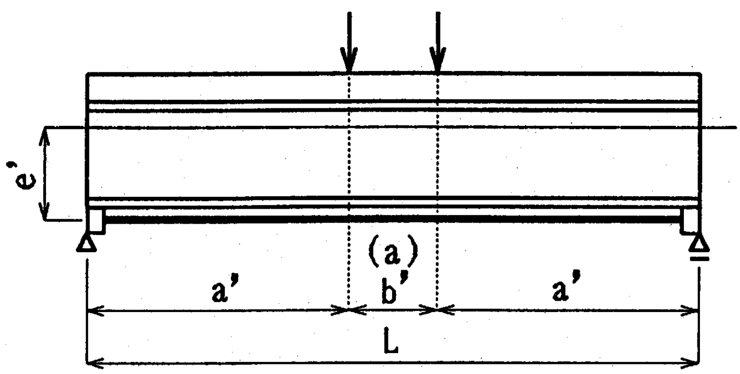


Fig. 3 Girder with Cable in the Straight Line Arrangement

where:
ε_s : the girder strain for the cable position at the load point,
ε_{sH} : the strain in the girder due to moments caused by eccentricity of the cable attachment position caused by the prestress,
L : the span length of the composite girder.

Furthermore, if the strain at the cable position after the strain increment ε_{s1} is taken to be ε_{sL}, it can be found from the following equation.

$$\varepsilon_{sL}=\frac{e'}{y_{s1}}\varepsilon_{s1} \tag{5}$$

Also, ε_{sH} can be found from:

$$\varepsilon_{sH}=\frac{E_t A_t d_t^2}{E_s I_v}\varepsilon_t \equiv C_1\varepsilon_t \tag{6}$$

where:
E_s, E_t : Young's modulus for the composite girder and the cable, respectively,
A_t : the cross sectional area of the cable,
I_v : the moment of inertia of the equivalent section for the composite girder.

Using Eqs. (5) and (6), ε_s can be expressed as follows.

$$\varepsilon_s=\varepsilon_{sL}-\varepsilon_{sH}=\varepsilon_{sL}-C_1\varepsilon_t \tag{7}$$

Substituting Eq. (7) in Eq. (4) and simplifying, the following equation is achieved.

$$\delta_s=(a'+b')\varepsilon_{sL}-L\cdot C_1\varepsilon_t \tag{8}$$

By substituting Eqs. (3) and (8) in Eq. (2), the cable strain for the straight line arrangement can be found from the following equation.

$$\varepsilon_t=\frac{(a'+b')\varepsilon_{sL}}{l+LC_1} \tag{9}$$

In the case of the queen post arrangement shown in Fig. 4, a change will occur in the girder strain at the cable position in both the girder spanwise and normal directions, therefore, the following three regions are considered:
Region ① $0 \leq x < a$,
Region ② $a \leq x < a'$,
Region ③ where the moment induced by the load is constant.

In where, if ε_{sH} is the strain in the girder due to moments caused by eccentricity of the cable attachment position caused by the prestress, ε_{sV} is the strain in the girder caused

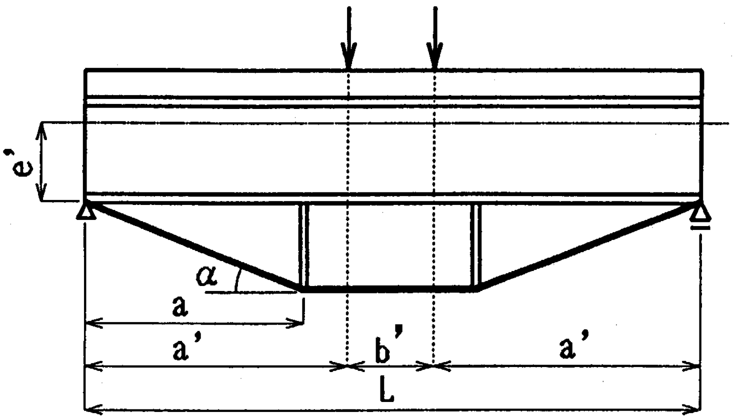


Fig. 4 Girder with Cable in the Queen Post Arrangement

by the vertical component of the prestress at the cable bend position, and ε_{sH0} is the strain of the girder at the cable attachment position, then ε_{sH}, ε_{sV}, and ε_{sH0} can be obtained from the following respective equations.

$$\varepsilon_{sH}=\frac{E_t \cdot A_t \cdot e' \cdot \cos\ \alpha \cdot d_t}{E_s I_V}\ \varepsilon_t \equiv C_1 \varepsilon_t$$
$$\varepsilon_{sV}=\frac{E_t \cdot A_t \cdot a \cdot \sin\ \alpha \cdot d_t}{E_s I_V}\ \varepsilon_t \equiv C_2 \varepsilon_t$$
$$\varepsilon_{sHO}=\frac{E_t \cdot A_t \cdot e' \cdot \cos\ \alpha \cdot e'}{E_s I_V}\ \varepsilon_t \equiv C_3 \varepsilon_t \tag{10}$$

[Region ① $0 \leq x < a$]
In this region, ε_{sL} and ε_{sV} at the cable position vary in both the spanwise and normal directions. The value of ε_{sH} varies only in the direction normal to the span. As a result, within this region, the girder deformation at the cable position can be obtained from the following equation.

$$\int_0^a \varepsilon_s dx=\int_0^a \frac{x}{a'}\frac{x}{a}\varepsilon_{sL}dx-\int_0^a \frac{x^2}{a^2}\varepsilon_{sV}dx-\int_0^a \left(\frac{x}{a}\varepsilon_{sH}+\varepsilon_{sHO}\right)dx \tag{11}$$

[Region ② $a \leq x < a'$]
In this region, ε_{sL} at the cable position varies in the composite girder spanwise direction. Also, ε_{sH} and ε_{sV} remain constant. Therefore, within this region, the girder deformation at the cable position can be obtained from the following equation.

$$\int_a^{a'} \varepsilon_s dx=\int_a^{a'} \frac{x}{a'}\varepsilon_{sL}dx-\int_a^{a'} (\varepsilon_{sH}+\varepsilon_{sV})dx \tag{12}$$

[Region ③ region of constant moment induced load]
In this region, the values of ε_{sL}, ε_{sH}, and ε_{sV} are all constant, so the deformation of the

girder can be obtained from the following equation.

$$\int \varepsilon_s dx = b'(\varepsilon_{sL} - \varepsilon_{sH} - \varepsilon_{sV}) \tag{13}$$

From the above, substituting Eq. (3) and Eqs. (10)~(13) in Eq. (2), the cable strain ε_t for the queen post arrangement can be obtained from the following equation.

$$\varepsilon_t = \frac{(a' - \frac{a^2}{3a'} + b')\varepsilon_{sL}}{l + (2a' - a + b')C_1 + (2a' - \frac{4a}{3} + b')C_2 + 2aC_3} \tag{14}$$

3) Stress and Internal Forces

For the strain given at the various positions within the cross section by the above equations, the stresses within the section due to the strain increment are calculated using the stress-strain relations for the various materials. By multiplying the various stresses with the cross sectional area, the internal forces in the various parts of the flange, web, and cable can be obtained[8].

4) Neutral Axis

For the equilibrium of forces acting on the PS girder, the distance of the lower end of the bottom flange to the neutral axis y_{s1}, can be found by satisfying the following equation.

$$F_T - F_C = 0 \tag{15}$$

where:
F_T : the sum of the tension forces acting on the cross section,
F_C : the sum of the compression forces acting on the cross section.

5) External Load and Deflection

The resisting moment due to the internal forces is found by summing all the internal moments about the previously determined neutral axis, and by equating this to the moment resulting from the externally applied load, the external load can be evaluated. Also, the elasto-plastic deflection of the prestressed composite girder can be found using a general conjugate beam method with the curvature distribution diagram for the elasto-plastic region[8].

3. VERIFICATION TESTS FOR THE ANALYTICAL RESULTS

3.1 Outline of Verification Tests

Five test specimens were prepared for the verification tests, four PS composite girders with the queen post cable arrangement (two girders with cable within the cross section

Table 1 Types of Specimen

Specimen	Cable arrangement	Type of Cable	Initial prestressing force (Total of 2 cables, tf)
A	No cable	—	—
B	Queen-post type(Draped) Inside cross section	Steel strand	70.0
C	Queen-post type(Draped) Outside cross section	Steel strand	17.0
D	Queen-post type(Draped) Inside cross section	Aramid fiber strand	70.0
E	Queen-post type(Draped) Outside cross section	Aramid fiber strand	17.0

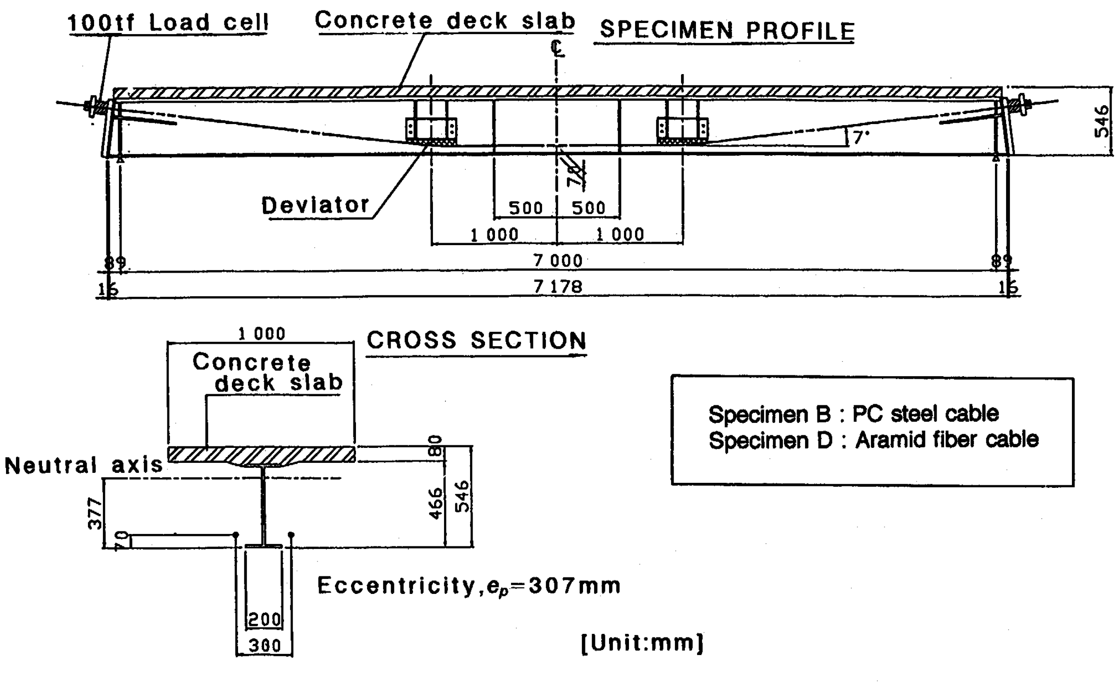


Fig. 5 Test Specimens (For Cables within the Cross Section)

and other two girders with cable arranged outside the cross section)[9], and a standard girder without cables. The specimens prepared were models of an actual composite steel plate girder bridge, the steel girder to concrete deck slab distribution ratio was made close to that of the actual bridge. As a result, the neutral axis within the elastic region of the girder lies just below the upper flange, and the cross section is very similar to the main girder cross section of a general composite steel plate girder bridge. Commercially available H section steel girder (SS400) was used[8], and two cable materials, PC steel cable and aramid fiber (braided cable). A 50mm thick steel plate ground to form a circular arc and PVC pipe were incorporated at the position where the cable bends to provide smooth cable deflection. An outline of the different

Table 2 Concrete Test Results

Specimen	Compressive strength (kgf/cm^2)	Modulus of elasticity (kgf/cm^2)
A,B	386	2.60×10^5
C,D	364	2.70×10^5
E	341	2.40×10^5

Table 3 Cable Material Properties from Material Tests

Cable material	Cross sectional area (mm^2)	Yield stress (kgf/mm^2)	Tensile strength (kgf/mm^2)	Modulus of elasticity (kgf/mm^2)
Steel strand (T21.8)	312.9*	$t_y=175$	$t_u=193$	$E_t=19{,}460$
Aramid fiber strand	127(1 cable)*	—	$t_{fu}=162$	$E_{tf}=5{,}982$

*The cross sectional area of the steel strand used in the analysis was $625.8mm^2$, Aramid braided cable cross section was $1{,}016mm^2$

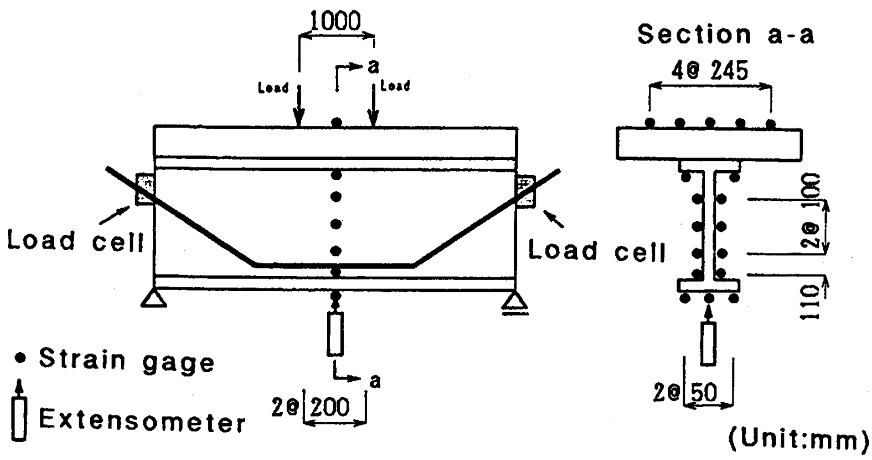


Fig. 6 Measurement Positions

types of specimens is given in Fig. 5 and Table 1. The concrete compressive strength obtained from material testing and the cable material characteristics are shown in Tables 2 and 3, respectively. The amount of prestress introduced was set to produce a tensile stress of $20kgf/cm^2$ in the upper surface of the concrete deck slabs. The introduction of the prestress via the cables was carried out with the specimen placed on a loading device incorporating two jacks to prevent out of plane bending moments before commencement of the test. The cable tension was measured using load cells situated at both ends of the cables. As shown in Fig. 6, the load was applied in the form of two point loads, the application of the load commencing as soon as the cables were tensioned. To prevent lateral buckling of the main girder, horizontal displacement of the upper flange of the girder was constrained. The measurements made during the

verification test were:
(1) span central deflection,
(2) span central strain distribution,
(3) prestress and increase in cable tension(The various measuring points are shown in Fig. 6).

3.2 Comparison of Test and Analysis Results

An example of a comparison of the test and analysis results can be seen in Figs. 7~11 and Table 4. Firstly, Table 4 shows the prestress introduced, the values are 4tf~7tf smaller than the target values for the specimens shown in Table 1. This was because when prestress was applied to the standard composite girder, the prestress was fixed while controlling the value of the strain gauge attached to the upper surface of the concrete deck slab to prevent cracking. For this reason, the results of the analyses for Figs. 7~11 and Table 4, were calculated using the actual prestress introduced into the test specimens. The yield of the PS composite girder initiated at the bottom edge of the

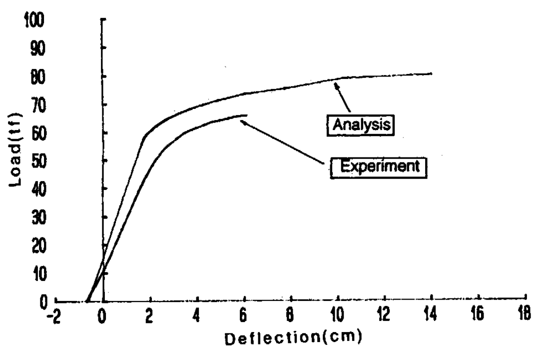


Fig. 7 Load~Midspan Deflection Curves (Specimen B, Cable within the Section, PC Steel Cable)

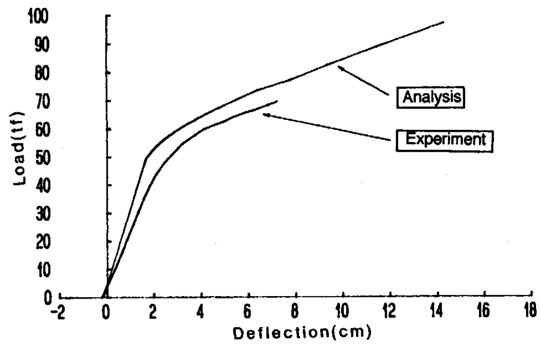


Fig. 8 Load~Midspan Deflection Curves (Specimen C, Cable outside the Section, PC Steel Cable)

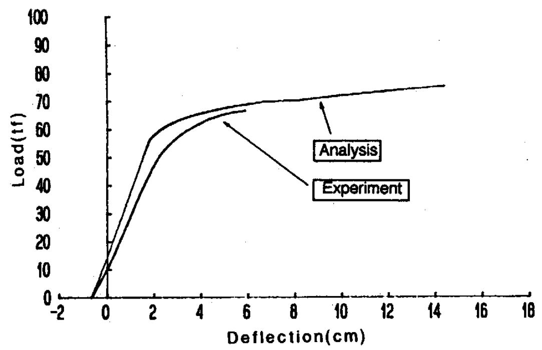


Fig. 9 Load~Midspan Deflection Curves (Specimen D, Cable within the Section, Aramid Fiber Cable)

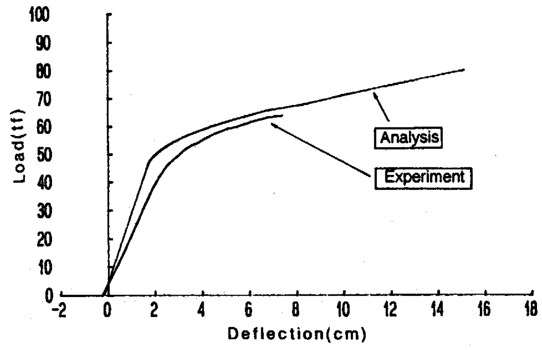


Fig. 10 Load~Midspan Deflection Curves (Specimen E, Cable outside the Section, Aramid Fiber Cable)

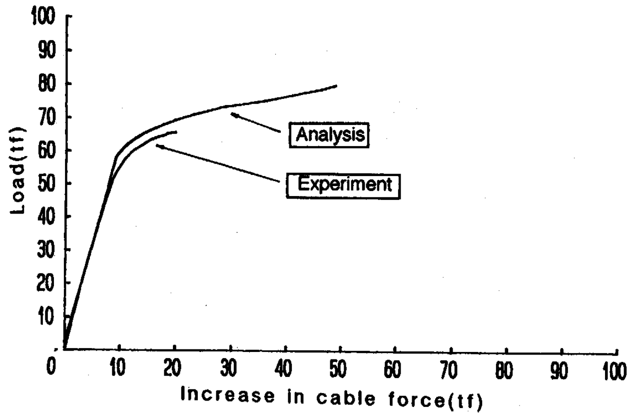


Fig. 11 Load~Increase in Cable Force Curves
(Specimen B, Cable within the Section, PC Steel Cable)

Table 4 Table of Experimental and Analysis Results

Specimen		Initial prestressing force (tf)	Yield load (tf)	Ultimate load (tf)
A: Standard girder	Experiment	—	29.8	48.8
	Analysis	12.9	46.0	63.7
B: Cable within section (Steel strand)	Experiment	63.3	56.2	76.9
	Analysis	63.3	52.2	65.8
C: Cable outside section (Steel strand)	Experiment	11.3	47.9	86.8
	Analysis	11.3	39.4	69.5
D: Cable within section (Aramid fiber strand)	Experiment	62.4	54.8	71.4
	Analysis	62.4	48.2	66.5
E: Cable outside section (Aramid fiber strand)	Experiment	—	37.7	49.6
	Analysis	12.9	46.1	77.1

lower flange in both the tests and in the analyses. Also, the ultimate failure of the PS composite girder, for all cases of test and analysis, occurred as the result of compressive failure of the concrete deck slab.

The tests were carried out varying the parameters of cable eccentricity (arranged within the cross section and outside the cross section), and cable material properties (steel and aramid fiber strand). According to the analysis, the predicted effects of the cable eccentricity on the prestressed composite girder are:
i) The yield load of the composite girder would be higher for cables within the cross section than for cables outside the cross section. In other words, the failure of the test

specimens in the current tests occur in the lower flange. For this reason, in order to increase the yield load of the girder, it would be necessary to introduce a prestressing force to apply a compressive pre-load in the lower flange to extend the elastic region. However, a large prestress can be introduced into the girder by making the cable eccentricity small. Therefore, cables within the cross section gives greater extension of the elastic region and results in a higher yield load than cables outside the cross section.

ii) The ultimate load for the composite girder with cables outside the cross section would be higher than for the case of cables within the cross section. In other words, with increasing cable eccentricity, the moments due to eccentricity of the applied prestress also become greater and thus results in a greater prestress effect. Also, the increase in cable tension, which is a statically indeterminate force, increases with increasing cable eccentricity. For these two reasons, the ultimate load for the case of cables outside the cross section is greater than for cables within the cross section. The above two effects can be clearly seen by comparison of the test results (Figs. 7～11 and Table 4) obtained for specimen B (steel strand, cable within the cross section) with those obtained for specimen C (steel strand, cable outside the cross section).

Next, considering the effects of the cable material properties on the composite girder, analysis predicts that the higher the elastic modulus of the material, the higher the values of yield load and ultimate load. For the case of the test specimens with steel strand, the cross section was chosen so that the cables would not yield before the steel girder yield, and would not break after the yield of the girder. In the case of aramid fiber strand too, the cross section was chosen so that even after the ultimate condition was reached for the girder, the cable would not break. For these reasons, only the cable elastic modulus has an effect on the girder stresses. This means that if the cable elastic modulus is large, the increase in cable tension will also be large, and as a result the girder yield load and ultimate load will also become large. These effects can be seen by comparison of the yield load for specimen B (steel strand, cable within the cross section) with the yield load for specimen D (aramid fiber strand, cable within the cross section), and comparison of the ultimate load for specimen C (steel strand, cable outside the cross section) with the ultimate load for specimen E (aramid fiber strand, cable outside the cross section) (Figs. 7～11 and Table 4).

Finally, the following observations can be made on overall comparison of the test results and analysis results:

i) Overall, good agreement was achieved for the behavior of the load-midspan deflection relation for the test specimens. However, the value of initial gradient for the test was lower than that obtained from analysis. This can be attributed to such factors as the effective width of the concrete deck slabs at the slab center and in the region of the support points being different.

ii) Small discrepancies occurred between the values of yield load and ultimate load, however, good agreement can be seen in such trends as the higher yield load being achieved in the case of cables within the cross section than outside the cross section, and lower values of yield and ultimate load occurring for low values of elastic modulus.

iii) Although differences occurred in the final values of increase in the cable tension due to the differences in ultimate load, good overall agreement was achieved between the test results and the results of the analyses. This indicates the validity of the analytical assumption made when obtaining values of cable strain, that the extension of the cable equals the extension of the girder at the cable position.

4. PERFORMANCE EVALUATION OF PRESTRESSED COMPOSITE GIREDR AND DESIGN CONCEPT OF STRENGTHENING

4.1 Evaluation of Performance of Prestressed Composite Girder by Parameter Analysis

Above, the applicability of the incremental deformation method as a method of analyzing the elasto-plastic behavior of PS composite girders was verified by test. As a result, it was found that the introduction of prestressing force via cables to the composite girder provided an increase in the yield load and ultimate load, and compared to the standard girder without cables, outstanding performance was demonstrated. This indicates that the incorporation of external cables provides a means of strengthening existing bridges. Incidentally, although it has not been demonstrated in this paper, it has been shown that besides the increases in yield and ultimate loads, good agreement can be achieved between analytical and experimental results for the failure modes of the PS composite girder, characteristics of increase in cable tension, and cross sectional internal stress distribution[10].

For the purpose of strengthening the existing bridges indicated below, the parameters having an effect on the mechanical behavior of the PS composite girder were selected, and by means of a parameter analysis, an evaluation of the performance of the PS composite girder was made in relation to improving the yield load, ultimate load, and flexural rigidity. An explanation of the various symbols of the analytical model used in the analysis and details of the cross section of the composite standard girder are given in Fig. 12.

① b/L : Length span ratio of the horizontal portion of the cable,

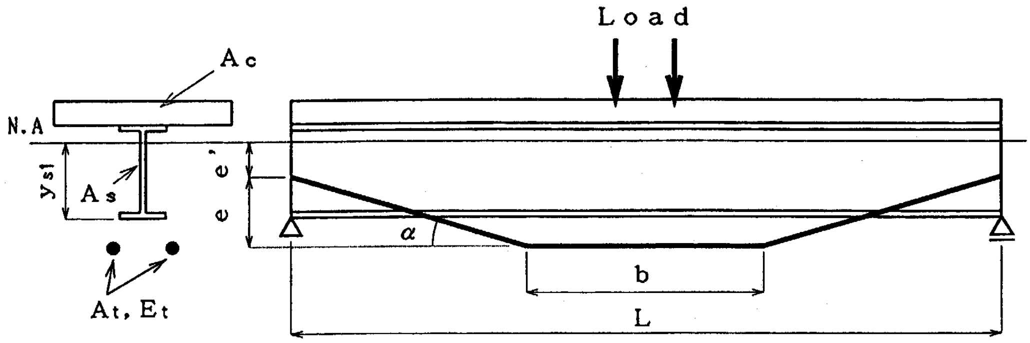
② e'/y_{s1}: Ratio of the eccentricity at the cable attachment position to the distance, from the lower surface of the bottom flange to the composite girder neutral axis,

③ e/y_{s1}: Ratio of the distance from the cable attachment position to the cable horizontal portion, to the distance from the lower surface of the bottom flange to the composite girder neutral axis,

④ E_t/E_s : Ratio of the Young's modulus of the cable to the Young's modulus of the steel,

⑤ A_t/A_V : Ratio of the cable cross sectional area to the composite girder cross sectional area,

⑥ A_c/A_s : Ratio of the concrete deck slab cross section to the steel girder cross section.



Deck slab	1000×80
Steel girder	H-450×200×9×14 (SS400)
Haunch	30
Span	7000

(Unit:mm)

Fig. 12 Model Symbols and Section Details used in Parameter Analysis

Here, the dimensionless parameters ①, ② and ③ are related to the arrangement of the cables. The dimensionless parameters ④ and ⑤ are related to the cable materials. The cable materials considered are generally FRP, the stress-strain relationship was assumed to vary linearly up to the point of failure[10]. The dimensionless parameter ⑥ relates to thickness of the deck slab, the width of the deck slab was made constant and only the thickness varied.

Of the results of the analysis to determine how the above parameters effect the improvement of such characteristics as yield load, ultimate load and flexural rigidity of the girder without cables (standard girder), main focus was placed on studying the effects of the cable arrangement. Here the yield load is defined as the load at which the yield strain is reached on the PS composite girder concrete deck slab upper surface, on the upper surface of the steel girder top flange, the lower surface of the steel girder bottom flange, or on the cable. Likewise, the ultimate load is defined as the load at which the ultimate strain limit is reached on the PS composite girder concrete deck slab upper surface, on the upper surface of the steel girder top flange, the lower surface of the steel girder bottom flange, or on the cable. In addition, the bending rigidity of the PS composite girder, EI, is defined in terms of ratio of the moment M_y and the curvature ϕ by the following equation.

$$EI = \frac{M_y}{\phi} \tag{16}$$

The terms used in the evaluation of the performance are as follows:

P_y/P_y' : Increase in yield load, here, P_y: PS composite girder yield load, P_y': standard girder yield load,

P_u/P_u' : Increase in ultimate load, here, P_u: PS composite girder ultimate load, P_u': standard girder ultimate load,

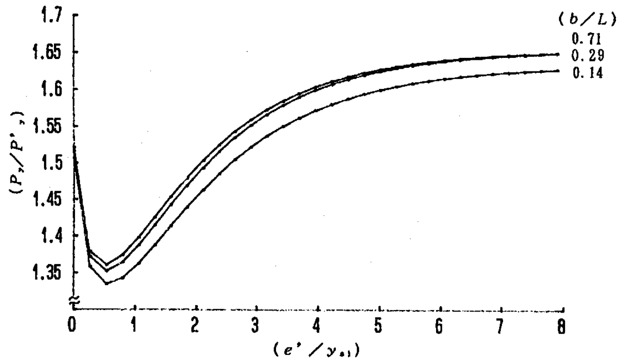


Fig. 13 Effects of (b/L) and (e'/y_{s1}) on Yield Load

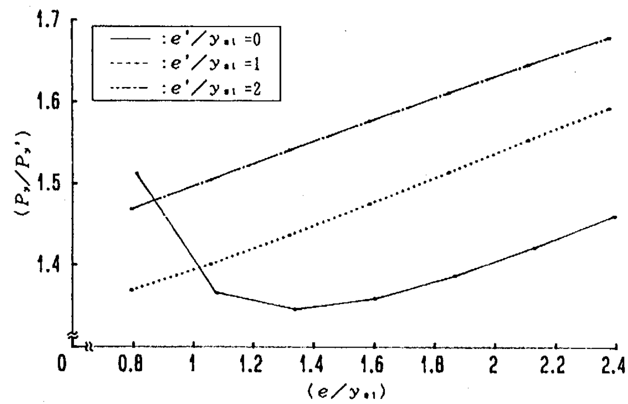


Fig. 14 Effects of (e'/y_{s1}) and (e/y_{s1}) on Yield Load

EI/EI': Increase in flexural rigidity, here, EI: PS composite girder flexural rigidity, EI': standard girder flexural rigidity.

Firstly, the influence of the cable arrangement on the yield load is shown in Figs. 13 and 14. Fig. 13 shows the case where the dimensionless parameter e/y_{s1} is held constant ($=0.81$), and e'/y_{s1} is varied by three different values of b/L. In Fig. 14, is shown the variation of e/y_{s1} for three values of e'/y_{s1}, with b/L held constant ($=0.29$) (Fig. 19). For Fig. 13 where $e'/y_{s1}=0$, that is for the case where the cable is attached at the neutral axis position, the increase in yield load in comparison with the standard girder is 1.50 times. After that, increase of the cable eccentricity resulted in the drop in this increase in yield load to a value of 1.35 times the standard girder. Further increases in the cable eccentricity caused the increase in yield load to rise again. The reason for this is that for the current analysis, in all cases the yield of the PS composite girder initiated in the bottom flange, so the increase in yield load depends on how much stress can be induced in the bottom flange in the direction opposite to the stress in the bottom flange caused by the load. As the maximum possible prestress was applied for each of the various cable arrangements (the value that causes the tensile stress in the upper surface of the deck slab to reach 20kgf/cm^2), arrangements with low eccentricity enable higher prestresses to be introduced. At the same time, the larger the eccentricity the larger the resulting moment at the cable attachment points, enabling effective introduction of stresses in the direction opposite to those caused by the applied load. In other words, the yield load drops in the region where the introduced prestress dominates the yield load increase, on the other hand, the yield load rises in the region where the eccentricity dominates. Also, further increases in eccentricity result in an increase in yield load, but when this value reaches around 1.60, no appreciable increase is seen after that point. Notwithstanding this, a value of e'/y_{s1} of $0 \sim 1.0$ can be considered practical, and a further increase in yield load within this range can be achieved by reducing the eccentricity and introducing a greater prestressing force. Three different values of the dimensionless parameter b/L were studied, but no significant difference was observed in the results, so it can therefore be concluded that the effect on the yield load is small. The increase in yield load for $e'/y_{s1}=0.81$ shown in Fig. 14 is equivalent to the increase shown in Fig. 13 for $e'/y_{s1}=0$, 1.0 and 2.0. Therefore, the increase for $e'/y_{s1}=0$ is greater than for the cases $e'/y_{s1}=1.0$ or 2.0. As shown in Fig. 14, if the value of e/y_{s1} is progressively increased, in the case of $e'/y_{s1}=0$ alone, the increase momentarily dropped and then continued to rise again. The reason is considered to be the same as for the case of Fig. 13. That is, the smaller the value of e/y_{s1} the greater the value of prestress that can be introduced, so increasing e/y_{s1} results in a momentary drop in the yield load increase. However, the larger the value of e/y_{s1} the larger the upward forces occurring at the saddle portion due to the prestress, so when the influence of upward forces at the saddle portion exceeds the influence of the prestress the increase in yield load will again rise. From the above, when the cable is attached at the neutral axis (the case of $e'/y_{s1}=0$), rather than increase e/y_{s1}, putting the cable within the girder cross section would be more effective in increasing the yield load.

The effect of the cable arrangement on the ultimate load can be seen in Figs. 15 and 16 (see also Fig. 19). The setting of the parameters in the various figures is the same as for the case of yield load. Firstly, in Fig. 15, an increase in the ultimate load results from increases in eccentricity (for constant $e/y_{s1}=0.81$). The reason for the difference in the trend from the case of yield load can be explained as follows: As the eccentricity becomes large, the increase in tension in the cable, that is a redundant force, also becomes relatively larger. The value of prestress that can be introduced becomes smaller as the eccentricity becomes large, but due to this increase in cable tension, in the region of the ultimate load where the deformation is large, the difference in prestress becomes small, and the effect of the eccentricity becomes larger, giving the results shown in Fig. 15. For the current analysis, the failure mode of the PS composite girder was, in all cases, by collapse of the deck slabs. Three different values of dimensionless parameter b/L were applied, but little difference was observed between them. The ultimate load, as with the yield load, being virtually unaffected. The same conclusions can be reached for Fig. 16 as for Fig. 15. The reason is that, when the distance in the girder depthwise direction, from the cable attachment to the cable horizontal portion(e), becomes great, the increase in prestress also becomes greater, so the difference in the introduced prestress, that is inversely proportional to the eccen-

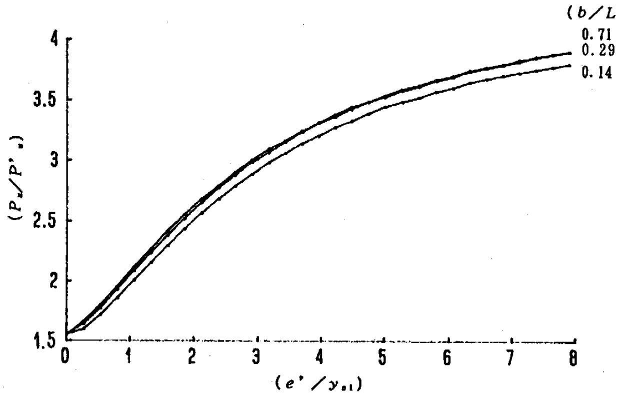


Fig. 15 Effects of (b/L) and (e'/y_{s1}) on Ultimate Load

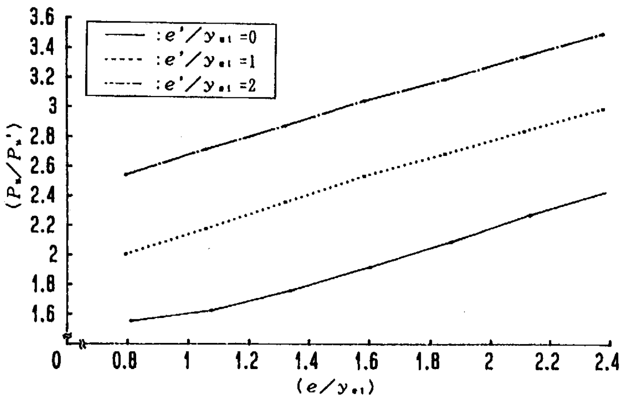


Fig. 16 Effects of (e'/y_{s1}) and (e/y_{s1}) on Ultimate Load

tricity occurring when the prestress is applied, becomes small in the vicinity of the ultimate load, and the effect of e becomes greater.

In Figs. 17 and 18 can be seen the effect of cable arrangement on the flexural rigidity (see Fig. 19). For the current analysis, the maximum possible prestress was applied to the composite girder for each of the cable arrangements. Therefore, the smaller the values of e'/y_{s1} and e/y_{s1}, the larger the prestress that can be introduced. However, as is clearly shown in Figs. 17 and 18, increase in the flexural rigidity accompanies the increases in e'/y_{s1} and e/y_{s1}. From this it can be stated that the introduced prestress has no effect on the increase in flexural rigidity. Conversely, when the eccentricity at the cable attachment position and the distance from the attachment position to the cable horizontal portion become great, the flexural rigidity increases. Three differing values of the dimensionless parameter b/L were applied, but for both the yield load shown in Fig. 13 and for the ultimate load shown in Fig. 15, there was little difference for the various values of b/L. The effect on the flexural rigidity of the PS composite

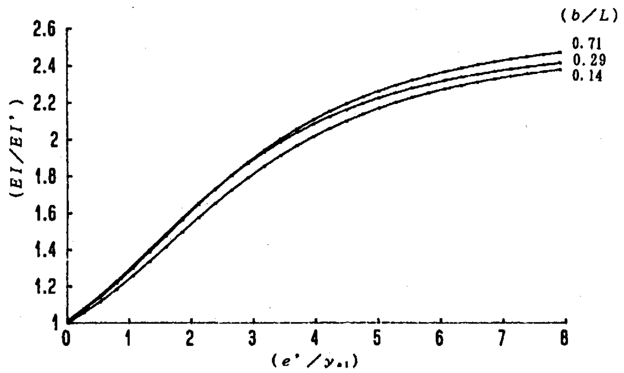


Fig. 17 Effects of (b/L) and (e'/y_{s1}) on Flexural Rigidity

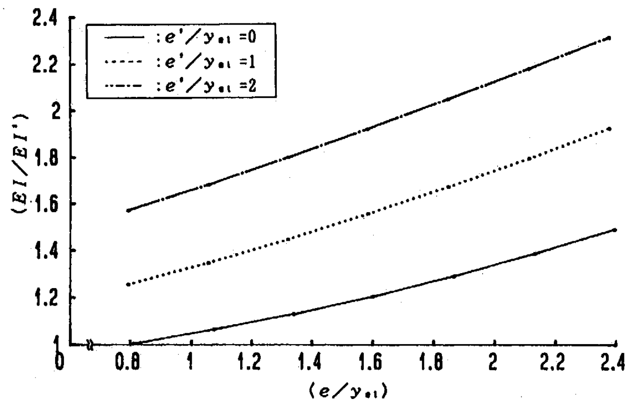


Fig. 18 Effects of (e'/y_{s1}) and (e/y_{s1}) on Flexural Rigidity

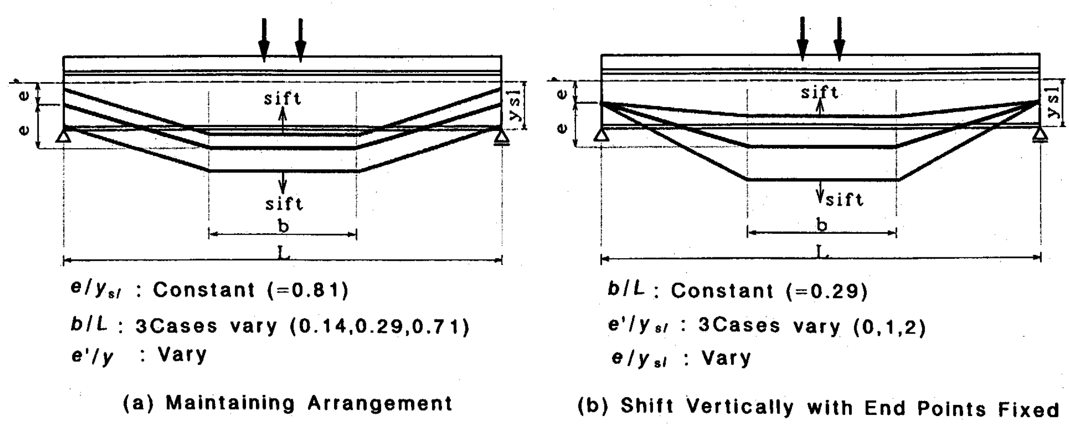


Fig. 19 Method of Varying Cable Arrangement

girder due to the cable arrangement can therefore be said to be negligible. In particular, for the practical range of values of e'/y_{s1} of $0\sim1.0$, as no appreciable differences occurred between any of the values of b/L, it is unreasonable to expect any increase in the PS composite girder flexural rigidity as the result of variations in b/L alone.

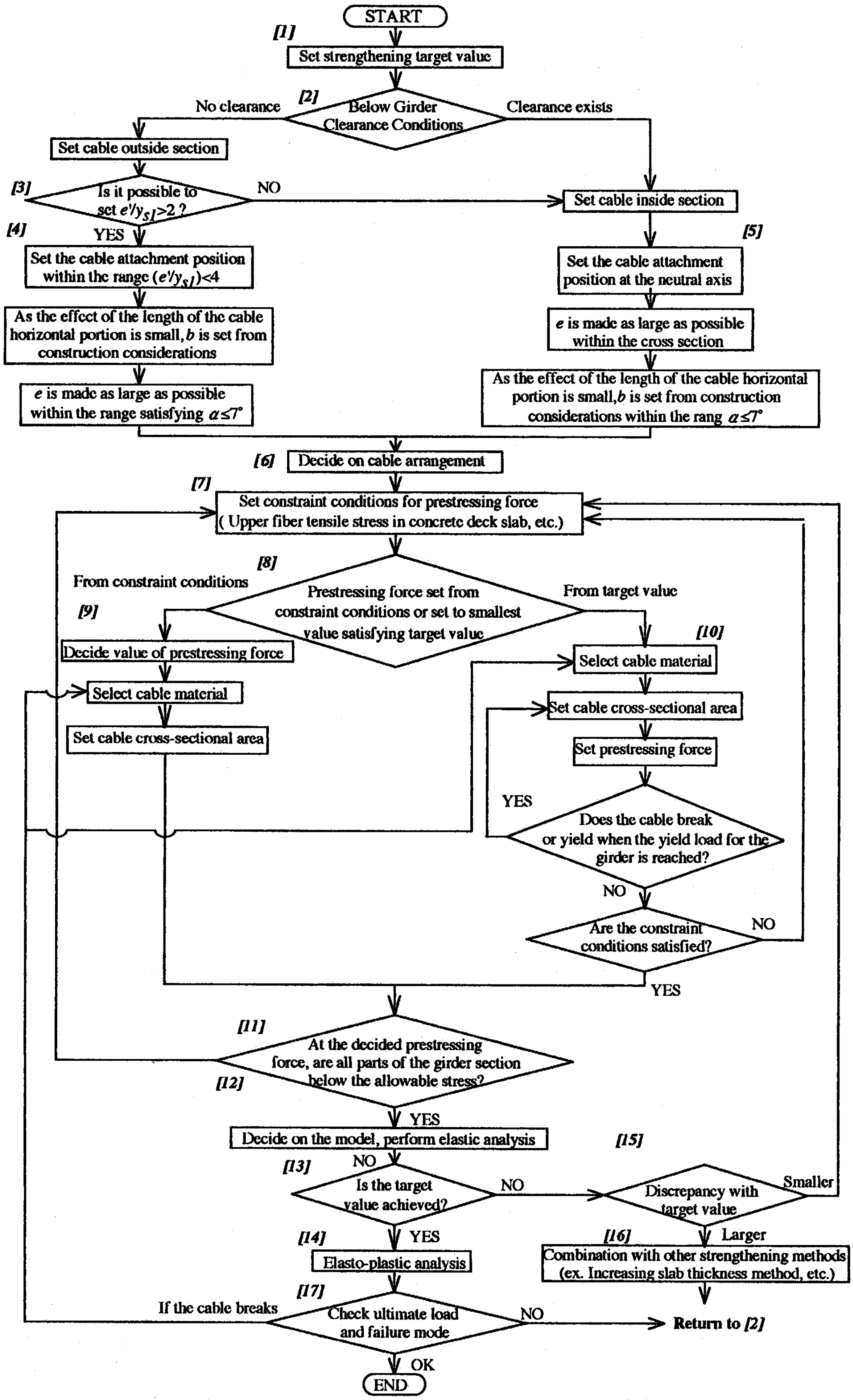


Fig. 20 Design Procedure for Strengthening Existing Bridges

4.2 Strengthening Design for Composite Girder Bridges Using Prestress

1) Procedure for Designing Strengthening Using Prestress
In this section will be described a method, based on the analysis in Section 4.1 above, of strengthening an existing bridge by increasing the girder yield load. Also, in order to demonstrate the applicability of the method, a practical example of its application to the composite girder model of an actual bridge will be given.

In Fig. 20 is shown the design procedure for strengthening by increasing the girder yield load. The queen post cable arrangement was adopted. Firstly, the actual target value of the strengthening (e.g. 1.5 times increase in yield load) is decided ([1] in Fig. 20). Next, depending on whether or not there is any restriction on the clearance below the girder, the choice is made whether to locate the cables within the cross section or outside to the cross section ([2] in Fig. 20). In other words, where outside location of the cables is possible, and in addition $e'/y_{s1} > 2$, enabling sufficient eccentricity (e') at the cable attachment position, from the results of the parameter analysis within the range $e'/y_{s1} < 4$ an effective increase in the yield load can be achieved. Here, the effect of the length (b) of the cable horizontal portion on the yield load is small compared with the effect of the eccentricity, so that it is decided with regard to consideration of such factors as construction of the bending portion ([3] and [4] in Fig. 20). On the other hand, in cases where there are restrictions on the clearance below the girder, or where a value of $e'/y_{s1} > 2$ cannot be achieved, the attachment of the cable will be at the girder end neutral axis, and the cable eccentricity (e) will be made as large as possible within the limits of the girder cross section ([5] in Fig. 20). A limit was placed on the cable angle (α) of $\alpha \leq 7°$. The reason for this is that in the case where FRP cable materials are used, it has been found that a loss of strength is suffered as a result of the bending process[11], and the restriction of $\alpha \leq 7°$ was applied to prevent such losses. As a result of the above, the cable arrangement is determined ([6] in Fig. 20).

As the next step, the cable material, the cable cross section, and the level of prestress to be introduced are decided. Firstly, the restraining conditions of the level of prestress have to be decided, here however, the case where the largest possible prestress satisfying the constraint conditions is adopted, and the case where the smallest possible prestress required to achieve the target strengthening is adopted, are both considered ([7] and [8] in Fig. 20). In the former case, the level of prestress is determined by consideration of the restraining conditions, after which the cable material and cable cross section are decided ([9] in Fig. 20). If FRP is included in the consideration of the material, as has been made clear by the results of tests and analyses, the larger the value of Young's modulus the greater the strengthening effect[10], however, the material is generally chosen with respect to such factors as the prevention of corrosion and the anchoring of the cables. The cable cross section is chosen so that cable yield (in the case of FRP cable breakage) will not occur at the time of girder yield. On the other hand, in the latter case, it becomes necessary to first decide on the value of Young's modulus and the cable cross section ([10] in Fig. 20). This is necessary because, the prestress is a redundant force, the cable tension will increase due to the deformation caused by increases in load, and in order to understand the relation between the load and increase in cable tension, the Young's modulus and cable cross section are required.

The prestress determined in this way satisfies the conditions set at [7] in Fig. 20, but no check has been made with regard to the stress condition of other regions within the cross section, so at this point the fact that the stress is within the allowable stress within the cross section is confirmed([11] in Fig. 20).

From the above, all the necessary values are decided, and using these values an elastic behavioral analysis is performed ([12] in Fig. 20). If the results of the analysis satisfy the target set at [1] above, the process proceeds to [14], if not, it proceeds to [15] ([13] in Fig. 20). If the target set at [1] above is satisfied within the elastic region, and the strengthening objectives have been achieved, an elasto-plastic behavioral analysis is performed, and a check of the safety made with regard to the ultimate condition ([14] in Fig. 20). Further, in cases where adequate strengthening cannot be achieved by the introduction of prestress alone, the use in combination with other methods must be considered, and the decision making carried out again starting with the choice of cable arrangement ([15] and [16] in Fig. 20). Also, in cases where the target is satisfied, but the ultimate load or mode of failure are different from planned [9], then it is necessary to return to [2] and re-evaluate the required parts ([17] in Fig. 20). Especially in the case of cable failure, it becomes necessary to return to the parts where the cable material and cross sectional area are decided and re-evaluate.

2) Example of the Application of the Strengthening Method Using Prestress

The following is an account of the trial application of this method to strengthen an actual bridge by the introduction of prestress.

The bridge chosen for this was designed as a second class bridge(TL-14 load), and was a simply supported, single span, live load composite girder bridge, having 3 main girders. With the increase in traffic volume in recent years, the live load on the bridge is expected to increase to a level equivalent to that of a first class bridge. For this reason, it was decided to raise the standard of the bridge to first class bridge standard by a combination of bridge widening, the introduction of prestress, and increasing the thickness of the deck slabs. The purpose of the strengthening of this bridge is therefore to raise the class of the bridge from second to first class, in other words to increase the yield load. In order to satisfy this objective, it should be possible to apply a TL-20 load (*TL-14 × 1.43 approx.*) to the bridge without the stress in any of the bridge members exceeding the maximum allowable stress. The target value for the strengthening was set to give a yield load after strengthening of *1.4 × the yield load* before strengthening. Also, the increase in the thickness of the concrete deck slabs was set at 8cm.

Below, the flow outlined in Fig. 20 is followed and the various values are determined.

Firstly, the cable arrangement is determined following the procedure from [2] in Fig. 20. As a result of the restraint condition that the clearance below the girder cannot be changed in the case of the bridge under investigation, the location of the cable attachment was set on the neutral axis prior to increasing the deck slab thickness. The cable eccentricity e was made as large as possible within the limits imposed by the fact that the cable must be contained within the main girder cross section. Taking into account the construction difficulties involved at the cable bending position, a cable bend at $L/3$ or $2L/3$ (L=span length) was chosen so as to clear the steel girder joint. In this case, the resulting cable bending angle was $\alpha=7°$ which satisfies the limit placed on this angle.

Next, the cable material, cable cross section, and the level of prestress to be introduced are determined. In the case of the previously described parameter analysis, the maximum prestress was determined with respect to the condition that the tensile stress in the upper surface of the deck slab did not exceed a certain value. However, in cases where a target value for the strengthening has been set, the prestress is taken as the lowest possible value that will satisfy this target. In this particular case the yield load must be increased by a factor of 1.43, so either the prestress that satisfies this target, or the prestress that satisfies the condition that the stress in the concrete deck slab upper surface does not exceed 20kgf/cm^2, the lower value of the two is used. Determination of the value of prestress is carried out using elastic analysis[8]. In determining the value of prestress, it is first necessary to apply the values of cable Young's modulus and cable cross sectional area. For the cable material there are many options including FRP, however, for the purpose of this example JIS standard PC steel strand was chosen for the evaluation. Although it is known from the parameter analysis that a large cable cross section results in a large yield load[10], out of safety and economic considerations, it was decided to adopt the smallest possible cross section satisfying the two conditions viz. that the cable will not have reached yield at the steel girder yield point, and also that the tensile stress induced in the cable when the prestress is introduced shall not exceed 40% of the tensile strength of the cable.

From the constraint conditions above, a prestress value of P_t=163tf, and a cable cross section of 21cm^2 were decided. This prestress satisfies the condition that the stress in the concrete deck slab upper surface is lower than 20kgf/cm^2, ($P_t \approx$ 170tf results in a tensile stress of about 13kgf/cm^2).

The various values determined above are shown in Fig. 21.

3) Confirmation and Consideration of the Strengthening Effect Achieved by Prestress
The results of an incremental deformation method analysis performed with regard to the composite steel girder before and after the application of prestress, are shown in Table 5 and Fig. 22. From Table 5, it can be seen that the increase in yield load by a factor of 1.43 was achieved. With regard to other girder characteristics, the ultimate load was increased by a factor of 1.88, while the elastic range of the relation between the load and deflection at center span showed an improvement of 1.19 times for the

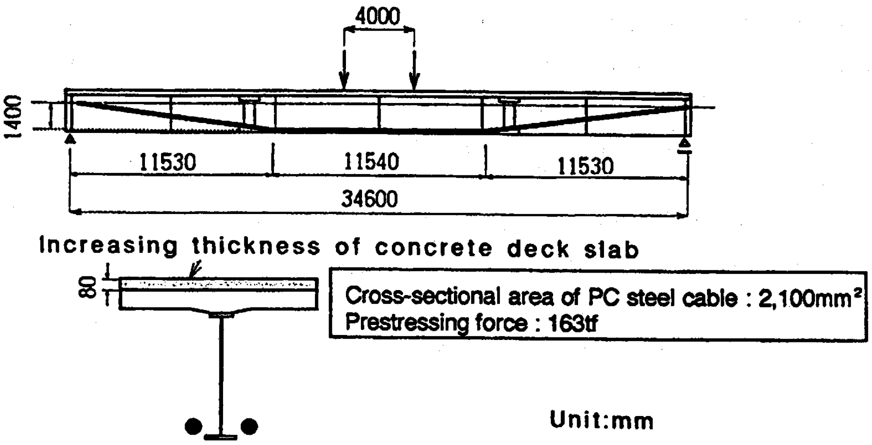


Fig. 21 Result of Strengthening Design by Prestressing

Table 5 Analytical Results of Strengthening Effect by Prestressing

	Before strengthening	After strengthening (Steel strand)
Yield load (tf)	54.6	78.5 (1.44 times)
Ultimate load (tf)	96.7	181.9 (1.88 times)
Gradient of linear portion (tf/cm)	11.0	13.1 (1.19 times)

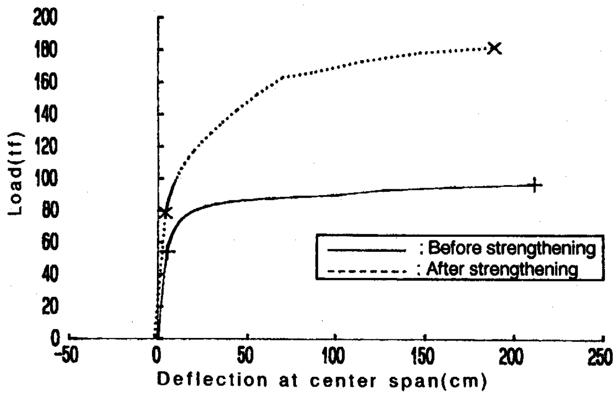


Fig. 22 Example of Performance Improvement after Strengthening (Load~Midspan Deflection)

gradient of the straight line portion of the curve. The mode of failure in both cases of the bridge before strengthening and after strengthening were the same, namely collapse of the concrete deck slab. The cable yielded when a load of 70tf was applied to the model after strengthening, however, with increased loading the cable reached the final condition without breaking, demonstrating the suitability of the determined cross section. Furthermore, from Fig. 22, it can be confirmed that the combination of deck slab thickening and introduction of prestress resulted in an extension of the elastic range, ultimate load and all the other prescribed objectives.

From the point of view of static behavior as considered above, it can be seen that the introduction of prestress provides an effective means of strengthening and improving the performance of the main girder. However, it is felt that the need remains to carry out an investigation into the dynamic behavior.

5. CONCLUSIONS

This research was carried out to investigate the effectiveness in improving the performance of composite girders by the introduction of prestress, as a method of strengthening existing bridge structures. The following is a summary of the results of this research.

① For the incremental deformation method mentioned in this paper, the strain of each part is incremented and the load is found that gives equal internal and external forces, the prestress, which is a redundant force, is found by satisfying the condition of compatibility for the deformation. This method makes it possible to evaluate the elasto-plastic behavior of the prestressed composite girder with relatively little computation, and its effectiveness was confirmed by the experimental results.

② From the experimental and analytical results it was found that the prestressed composite girder has yield load and ultimate load superior to the standard girder, and that the performance is effected by the choice of cable arrangement and cable material.

③ With the objective of strengthening existing bridges, various dimensionless parameters were varied and an analysis made of the effects of these various parameters on the performance of the composite girder. As a result, it was found that none of the parameters caused a decrease in composite girder performance, but the effectiveness of each parameter in increasing performance varied. It is necessary to select such parameters as the cable arrangement and cable material to suit the strengthening objectives. In particular, as the effect of the cable eccentricity at the cable attachment on the yield load is different to the effect on the ultimate load, adequate consideration is necessary.

④ A procedure was presented, based on a parameter analysis, for the strengthening of existing bridges by introducing prestress to improve the yield load. A model was made based on an actual bridge in need of yield load improvement, and the strengthening method applied following the proposed procedure. The objectives were successfully achieved and the efficiency of this strengthening method was adequately confirmed.

ACKNOWLEDGEMENTS

The authors would like to express their gratitude to all those who helped and gave their kind cooperation during the course of this research. In particular, to the Composite Structures Sub-committee of the Kansai Road Association(Chairman: Manabu

FUJII, Prof., Kyoto University), on whose work a part of this research was based. In addition, we would like to express our special thanks to all those at the Yagishita Laboratory at Kinki University, who helped in the confirmation trails.

REFERENCES

[1] Troitsky, M.S., "Prestressed Steel Bridges-Theory and Design", Van Nostrand Reinhold Company, New York, pp.1-16, 1990.

[2] Saadatmanesh, H., Albrecht, P., and Ayyub, B.M., "Guidelines for Flexural Design of Prestressed Composite Beams", Jour. of Str. Eng., ASCE, Vol.115, No.11, pp.2944-2961, November 1989.

[3] Hoadley, G., "Behavior of Prestressed Composite Steel Beams", Jour. of Str. Eng., ASCE, Vol. 89, No.ST3, pp.21-34, June 1963.

[4] Ayyub, B.M., Sohn, Y.G., and Saadatmanesh, H., "Prestressed Composite Girders under Positive Moment", Jour. of Str. Eng., ASCE, Vol.116, No.11, pp.2931-2949, November 1990.

[5] Japan Society of Civil Engineers, "Design Guidelines for Steel/Concrete Composite Structures", pp.80-85, March 1989.

[6] For example: Yamaguchi, T., Shiraki, B., and Adachi, Y., "Consideration of the Results of Loading Tests Performed on a Ikada-basi Prestressing Bridge", Mitsubishi Heavy Industries Research Report, No.883, pp.1-10, August 1958.

[7] Japan Society of Civil Engineers, "Standard for Prestressed Concrete", pp.35-44, January 1979.

[8] Miyamoto, A., Hirata, K., Yagishita, F., Mori, T., "Nonlinear Analysis of Prestressed Composite Girder and Its Application to Bridge Strengthening", Journal of Structural Engineering(JSCE), Vol.40A, pp.1101-1114, March 1994.

[9] Miyamoto, A., Yagishita, F., Maki, A., and Kobukata, Y., "Application of Prestressing Technique for Bridge Strengthening", Developments in Short and Medium Span Bridge Engineering '94, Nova Scotia, Canada, pp.1231-1245, August 1994.

[10] Kansai Road Association, Road Bridge Study Committee,"Report of Composite Structures Sub-committee, -Bridge Materials / Composite Structures-", pp.109-190, March 1994.

[11] Maruyama, T., Honma, M., Okamura, H., "Experimental Research into the Tensile Strength of FRP Rod During Bending", Proceedings of the 12th Annual Meeting of the Japan Concrete Institute (JCI), Vol.12-1, pp.1025-1030, June 1990.