# Cable-Stayed Bridges

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*INTERNATIONAL CONFERENCE ON CABLE-STAYED BRIDGES*
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# ANALYSIS OF SINGLE-PLANE FAN-TYPE CABLE-STAYED BRIDGES SUBJECTED TO NONUNIFORM SUPPORTS SETTLEMENTS

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## SUMMARY

A nonlinear analysis method is presented for single-plane fan-type cable-stayed bridges which are going to be constructed on soft clay and expected to be subjected to nonuniform supports settlements. The method is applied to the analysis of a model bridge whose structural dimensions are almost as same as those of Chao-Phya bridge. Examined with emphasis are the effects of out-of-plane rotations of towers around the longitudinal direction of bridge, on member forces, stresses of deck plates and so on.

## INTRODUCTION

The developments of both structural analysis techniques and design-fabrication-construction techniques based on highly developed computers have enabled us to construct long-span cable-stayed bridges.

In some cases, under various conditions such as social, economical, and aesthetic ones, it is needed to consider the construction of long-span single-plane cable stayed bridges on soft clay grounds. For this situation, the influences of nonuniform supports settlements on the states of stresses of various parts of superstructure must be examined beforehand in order to make appropriate design and construction as well as maintenance. Among them most important may be the investigation of the influence when the supports settlements cause rotation of towers around the longitudinal direction of the bridge. So far, no single-plane cable-stayed bridge has been constructed, besides Chao-Phya bridge, which has a main span as long as 400 meters or more. And also, there are few analytical results on the effects of out-of-plane displacements.

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8. Hasegawa, A., Iwakuma, T., Liyanage, K. K. and Nishino, F., 'A Consistent Formulation of Trusses and Non-Warping Beams in Linearized Finite Displacements', Structural Engineering/Earthquake Engineering, Vol.3, No.2, October 1986, pp. 477s-480s. (Proc. of JSCE, No. 374/I-6)

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10. CADAM Inc., CADAM-20 Geometry Interface, Lockheed Corp., California, December 1981.

11. Nishino, F., Hartono, W., Fujiwara, O., and Karasudhi, P., 'On Tracing Bifurcation Paths of Geometrically Nonlinear Structures', Structural Engineering/Earthquake Engineering, Vol.4, No.1, April 1987, pp.11s-17s. (Proc. of JSCE, No. 380/I-7)

12. See, T., and McConnel, R. E., 'Large Displacement Elastic Buckling of Space Structures', Journal of Structural Engineering, ASCE, Vol. 112, No. 5, May 1986, pp. 1052-1069.

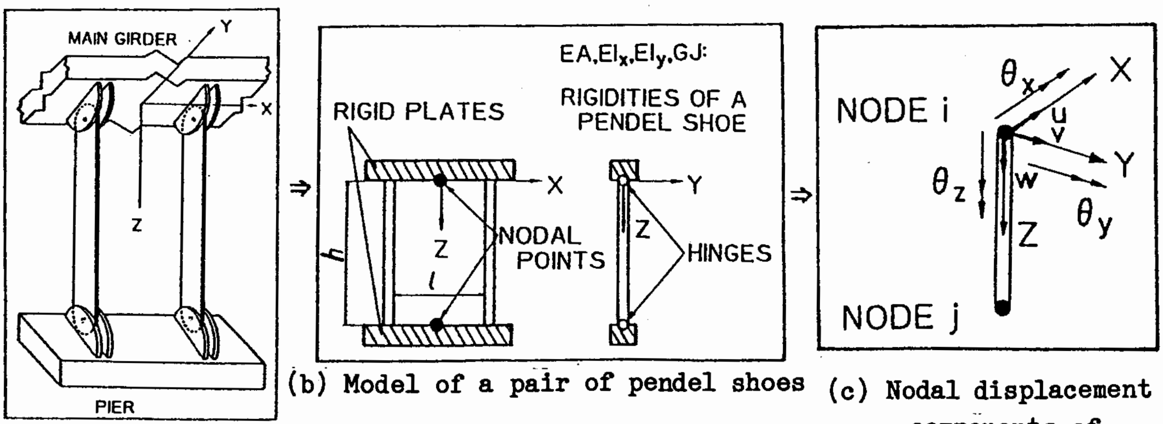
In this paper, a simplified geometrically nonlinear analysis method is presented for single-plane fan-type cable-stayed bridges which are going to be constructed on soft clay and expected to be subjected to nonuniform supports settlements. Then, the method is applied to the analysis of a model bridge whose structural dimensions are almost the same as those of Chao-Phya bridge. Examined with emphasis are the effects of out-of-plane rotations of towers on member forces, stresses of deck plates and so on.

The method is composed of two parts. Firstly, main girders and towers are represented by conventional frame elements with six degrees of freedom per each node. Cables are represented by rod elements with three degrees of freedom per each node, namely, three translational displacement components. For a pair of pendel shoes, a pendel shoe element is introduced which has six degrees of freedom per each node similar to the frame elements, but is slightly differently derived. These elements are applied to the geometrically nonlinear analysis method of space frames developed by the authors[1]-[3], which is capable of dealing with such problems that finite rotations occur around different axes in the three dimensional space. Thus, in-plane and out-of-plane behaviours are evaluated in a unified manner. Secondly, resulting member forces are utilized to calculate stresses at specified points on arbitrary sections of any members on the basis of thin walled members theory.

## ANALYSIS METHOD

### Geometrical nonlinear analysis method and method of structural modelling

In this paper, an analysis method composed of two parts is adopted as mentioned in the INTRODUCTION. Firstly, all the members except pendel shoes are modelled by ordinary space frame members with appropriate end conditions. The end condition of cables, for example, is free from rotation around any axes. A pair of pendel shoes, indicated in Fig.1, which connect the main girder and a pier is also modelled as a space frame member with six degrees of freedom per each node, but is derived slightly differently from the ordinary one. The stiffness matrix of this pair of pendel shoes elements is shown in Fig.2, where E and G are Young's modulus and shear modulus of elasticity, respectively. A, Ix, Iy, and GJ are area, moment of inertia around x-axis,




(a) A pair of pendel shoes

(b) Model of a pair of pendel shoes

(c) Nodal displacement components of a pendel-shoe-element

Fig.1 A pendel-shoe-element

$$
\begin{Bmatrix} X_i \\ Y_i \\ Z_i \\ M_{xi} \\ M_{yi} \\ M_{zi} \\ X_j \\ Y_j \\ Z_j \\ M_{xj} \\ M_{yj} \\ M_{zj} \end{Bmatrix}
=
\begin{bmatrix}
\frac{24EI_y}{l^3} & 0 & 0 & 0 & \frac{12EI_y}{l^2} & 0 & -\frac{24EI_y}{l^3} & 0 & 0 & 0 & \frac{12EI_y}{l^2} & 0 \\
0 & \frac{24EI_x}{l^3} & 0 & -\frac{12EI_x}{l^2} & 0 & 0 & 0 & -\frac{24EI_x}{l^3} & 0 & -\frac{12EI_x}{l^2} & 0 & 0 \\
0 & 0 & \frac{2EA}{l} & 0 & 0 & 0 & 0 & 0 & -\frac{2EA}{l} & 0 & 0 & 0 \\
0 & -\frac{12EI_x}{l^2} & 0 & \frac{8EI_x}{l} & 0 & 0 & 0 & \frac{12EI_x}{l^2} & 0 & \frac{4EI_x}{l} & 0 & 0 \\
\frac{12EI_y}{l^2} & 0 & 0 & 0 & \frac{2h^2EA}{l}+\frac{8EI_y}{l} & 0 & -\frac{12EI_y}{l^2} & 0 & 0 & 0 & -\frac{2h^2EA}{l}+\frac{4EI_y}{l} & 0 \\
0 & 0 & 0 & 0 & 0 & \frac{2GJ}{l} & 0 & 0 & 0 & 0 & 0 & -\frac{2GJ}{l} \\
\frac{24EI_y}{l^3} & 0 & 0 & 0 & -\frac{12EI_y}{l^2} & 0 & \frac{24EI_y}{l^3} & 0 & 0 & 0 & -\frac{12EI_y}{l^2} & 0 \\
0 & -\frac{24EI_x}{l^3} & 0 & \frac{12EI_x}{l^2} & 0 & 0 & 0 & \frac{24EI_x}{l^3} & 0 & \frac{12EI_x}{l^2} & 0 & 0 \\
0 & 0 & -\frac{2EA}{l} & 0 & 0 & 0 & 0 & 0 & \frac{2EA}{l} & 0 & 0 & 0 \\
0 & -\frac{12EI_x}{l^2} & 0 & \frac{4EI_x}{l} & 0 & 0 & 0 & \frac{12EI_x}{l^2} & 0 & \frac{8EI_x}{l} & 0 & 0 \\
\frac{12EI_y}{l^2} & 0 & 0 & 0 & -\frac{2h^2EA}{l}+\frac{4EI_y}{l} & 0 & -\frac{12EI_y}{l^2} & 0 & 0 & 0 & \frac{2h^2EA}{l}+\frac{8EI_y}{l} & 0 \\
0 & 0 & 0 & 0 & 0 & \frac{2GJ}{l} & 0 & 0 & 0 & 0 & 0 & \frac{2GJ}{l}
\end{bmatrix}
\begin{Bmatrix} u_i \\ v_i \\ w_i \\ \theta_{xi} \\ \theta_{yi} \\ \theta_{zi} \\ u_j \\ v_j \\ w_j \\ \theta_{xj} \\ \theta_{yj} \\ \theta_{zj} \end{Bmatrix}
$$

$$
= 2\,[K_p] + \frac{2h^2EA}{l} \cdot
\begin{array}{c c}
 & \text{column} \\
 & \begin{array}{ccccc} & 5 & & 11 & \end{array} \\
\text{row} \begin{array}{c} 5 \\ 11 \end{array} & \begin{bmatrix} \cdots & 1 & \cdots & -1 & \cdots \\ \cdots & -1 & \cdots & 1 & \cdots \end{bmatrix}
\end{array}
$$

; $[K_p]$ : a stiffness matrix of one pendel shoe as a ordinary bar member

$Y_i = M_{xi} = Y_j = M_{xj} = 0$ : end conditions

Fig.2 Stiffness equation of a pendel-shoe-element(Fig.1)

that around y-axis and torsional rigidity, respectively, of a cross section of the pendel shoe. And a pair of pendel shoes, whose height is h, is separated with a distance of l. Rigid bars are placed between the main girder's neutral axis and cable anchors, and also between the neutral axis and pendel shoes, to account for eccentric distances. Thus, whole the structure of a single-plane fan-type cable-stayed bridge is considered as a framework occupying a vertical plane.

Above mentioned elements are introduced into the geometrically nonlinear analysis method developed by the authors[1]-[3]. The validity of the method has been confirmed through its many applications to example problems. Nodal displacements and member forces, or stress resultants, are then calculated by this analysis method.

Secondly, main girders and towers are considered as thin walled members. Stresses at specified points of any section are obtained using the results given in the first process.

## Effects of dead loads on superstructure responses to live loads

After the completion of construction, each members are acted by dead loads. In the case of flexible structures such as cable-stayed bridges, the effects of dead loads may be nonlinear. So if a strict analysis is required, the construction process must be faithfully simulated. To examine whether this simulation is needed or not, some calculations are performed and the results are compared in advance for a sample structure.

a) Analyses of complete system and psudo-complete system

In a real structure, along with the progress of construction, member forces are induced for structural elements already constructed, and these member forces increase according to additional advancement of the construction. Therefore, though it's a matter of course, the structural configuration after the completion is formed under the action of dead loads. The structural system with this configuration is referred here to complete system. Whereas

**Table 1** Effects of dead loads on superstructure responses to live loads

| CASE | 0 | 1 | 2 | 3 |
|---|---|---|---|---|
| SYSTEM | PSEUDO-COMPLETE SYSTEM | COMPLETE SYSTEM | PSEUDO-COMPLETE SYSTEM | PSEUDO-COMPLETE SYSTEM |
| LOADS | DEAD LOADS ONLY | LIVE LOADS | LIVE LOADS ONLY | DEAD LOADS AND LIVE LOADS |
| SCHEMATIC DESCRIPTION OF SYSTEM AND LOADS |  |  PRESTRESSED BY DEAD LOADS |  |  |
| RESULTS OF EXAMPLE CALCULATIONS FOR HALF SPAN UNIFORM LOADING WITH AN INTENSITY OF 8.25 tf/m. A MODEL BRIDGE DESCRIBED IN FIG.4 IS USED. | | | | |
| DISPLACEMENT AT THE CENTER OF THE MAIN SPAN (m) | 1. 3 3 8 | 0. 6 5 3 | 0. 6 5 2 | 1. 9 9 7 |
| MAXIMUM DISPLACEMENT (m) | 1. 3 3 8 | 0. 8 5 4 | 0. 8 5 0 | 2. 0 9 2 |

a structural system which takes the same configuration with that of the complete system but is not loaded in dead loads is referred to pseudo-complete system. Analyses 1) of complete system under a certain live load, and 2) of pseudo-complete system under the same live load, 3) of pseudo-complete system under the combined action of the dead loads and the live load, together with 0) an analysis of pseudo-complete system under the dead loads only are conducted to investigate the effects of the dead loads on the response characteristics of the superstructure to live load actions(Table.1).

Comparing the results for the cases 1) and 2), we can find little difference in the responses to live loads as indicated in Table.1 (The example structure is modelled after Chao-Phya bridge (Fig.3), though there are several modifications.). And also, it can be found that the results of case 3) is almost same as the sum of the results of cases 0) and 2). So it is concluded the effect of prestress induced by dead loads on the superstructure responses to live loads is quite small in case of actual cable-stayed bridges. Thus, concerning to displacements, the results corresponding to the pseudo-complete system can be considered as those of the complete system itself. As to stress resultants, the results corresponding to the pseudo-complete system added by the prestresses introduced by dead loads are considered to approximate quite well those of the complete system.

b)Simplified calculation method for the complete system

To simulate each construction stages, adjustment of cable length at each stages must be considered in equilibrium equations. Calculations of prestressed structures, which are going to have additional members as well as additional loads, must also be performed. But when only the complete system are to be considered as is the case now, the above mentioned calculating processes are not necessarily needed. The following simplified calculation will be sufficient enough : firstly,pseudo-complete system is analyzed under dead loads to give stress resultants as shown in Table 1- 0),which are considered in turn as prestresses in the second analysis of pseud-complete system subjected to live loads (Table 1- 1)). The reason why this simplified calculation is sufficient is because it is expected that inequilibrium force in the pseudo-complete system caused by the introduction of the stress resultants which satisfy the equilibrium conditions of pseudo-complete system after the deformation is negligibly small under the condition of small deformation. The results shown in Table 1- 1) is obtained by this simplified calculation.

## APPLICATION OF THE METHOD TO A MODEL BRIDGE

### A model bridge - structural dimensions and boundary conditions

In order to obtain practical results, an actual long-span single-plane fan-type cable-stayed bridge is selected to be modelled here. Namely, as a model bridge, we choose Chao-Phya bridge (Fig.3)[4]. The bridge is modelled as shown in Fig.4 where principal structural dimensions, boundary conditions, and element division are illustrated. Cross sectional dimensions and material properties are also set after the real bridge.

### Analysis of in-plane responses to in-plane loadings

Here, among several calculations conducted, the results of the following two cases are shown. In the following, a symbol tf is used to indicate the unit tonf.

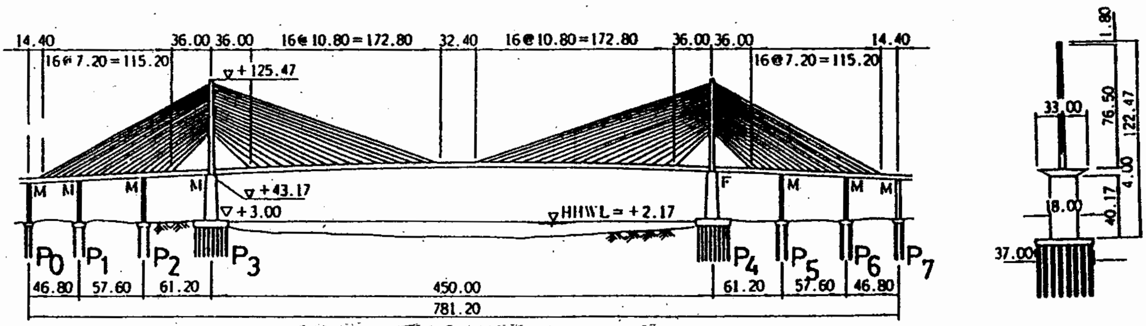
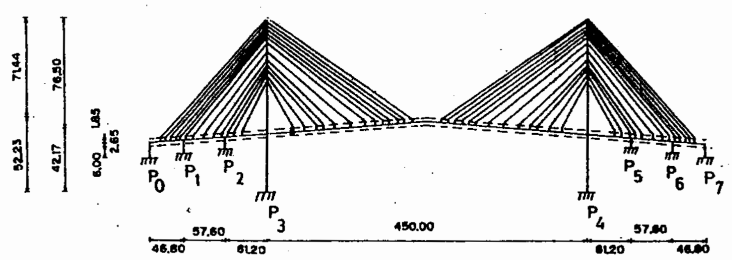
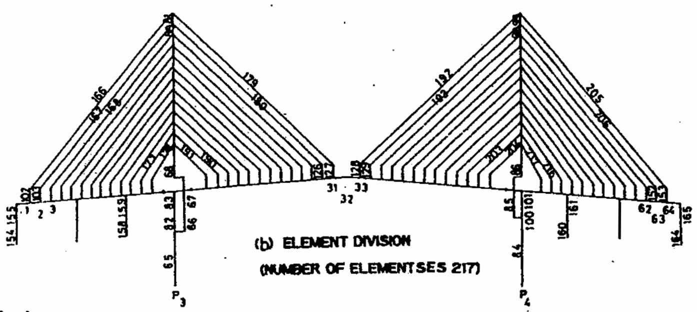


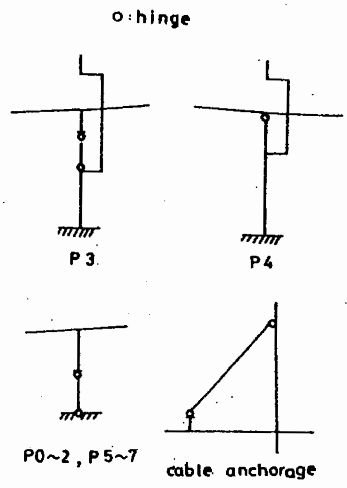

Fig.3 Chao-Phya Bridge[4]




(a) Structural model used in in-plane analyses

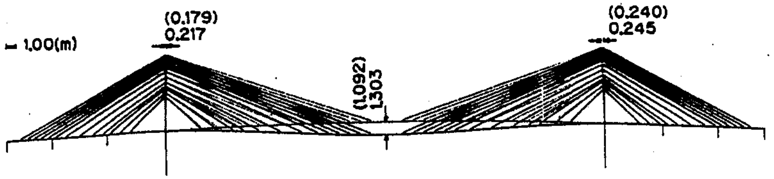



(b) Element division (Number of element is 217)

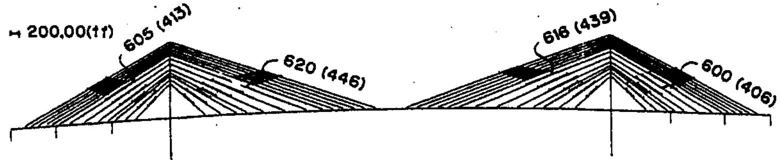



(c) Boundary conditions and member end conditions

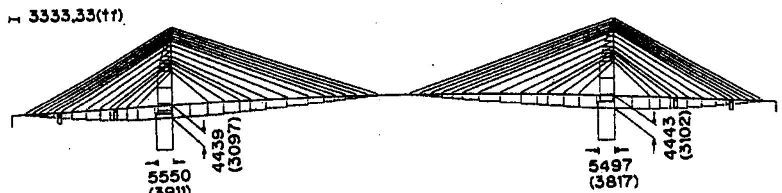
Fig.4 Structural model and element division for in-plane analyses



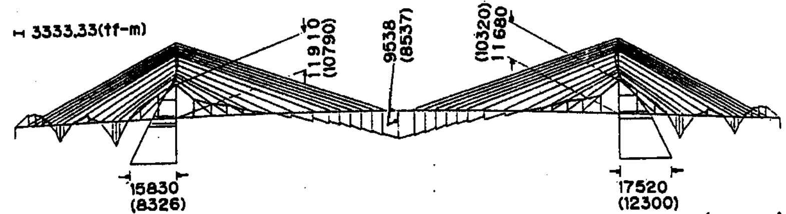

(a) Deformed configuration (m)




(b) Axial forces in cables (tf)

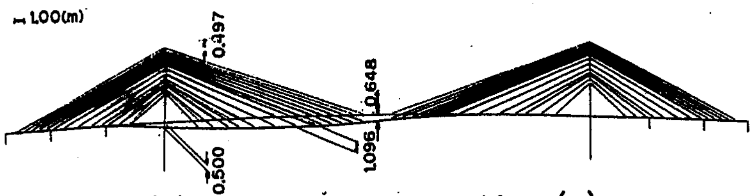



(c) Axial forces in main girders and towers (tf)

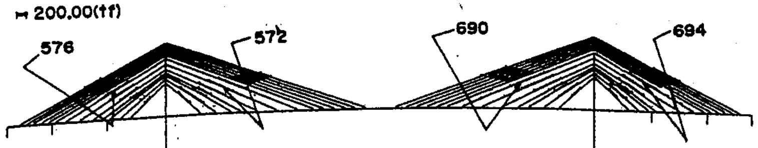



(d) Bending moments in main girders and towers (tf-m)

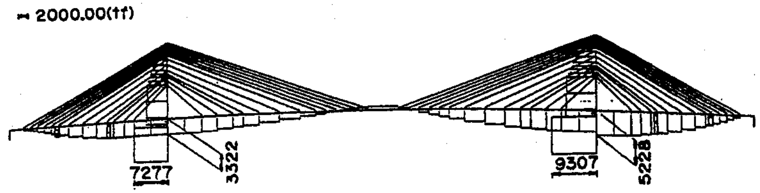
Fig.5 Results of an in-plane analysis
Uniform loading in the main span
with an intensity of 8.25 tf/m



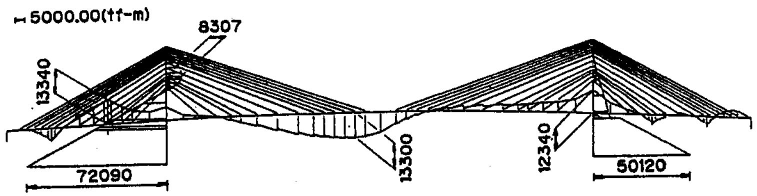

(a) Deformed configuration (m)




(b) Axial forces in cables (tf)




(c) Axial forces in main girders and towers (tf)




(d) Bending moments in main girders and towers (tf-m)

Fig.6 Results of an in-plane analysis
In-plane support settlement of
pier P3 by 0.5 meters

a)Case-1 : Loading in the main span with 8.25 tf/m uniform load

Among the calculated results the followings are illustrated in Fig.5., where calculated values at typical points are placed for references;
Fig.5(a) : deformed configuration (deformation is scaled up) [unit:m],
Fig.5(b) : axial forces in cables [unit:tf],
Fig.5(c) : axial forces in main girders and towers [unit:tf]
Fig.5(d) : bending moments in main girders and towers [unit:tf-m].
We have confirmed, though evidence is not shown here, that these results correspond well with the values given in the design paper of Chao-Phya bridge within reasonable accuracy, under the condition that there are differences in structural modelling between the design calculations and present analysis.

b)Case-2: In-plane support settlements of pier P3 (on which stands a tower)

The relation between P3 (pier P3) support settlement and deflection of main girder at center section, as well as that between the former and the maximum axial force in cables are found almost linear, up to the settlement of 0.5 meters, which is just a reference value. Within the range of 0.5 meters settlement, nonlinearity induced in this case can be regarded less than 5%. So, in Fig.6 only the results at the final step - 0.5 meters settlement - are presented, the figures show deformed configuration and stress resultants diagrams.

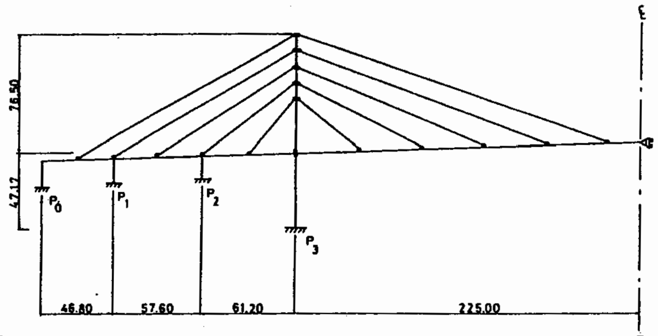
The comparison between the results shown in Fig.6 and the one obtained for the case where the complete system is at its rest, namely, subjected to dead loads only, tell us the followings : when support settlement of 0.5 meters generates at P3 in complete system, stress resultants increase from those values under the dead loads only, by 14% for the maximum axial cable force, by 12% for the maximum axial force in main girders, by 10% for the maximum axial force in towers, by 35% for the maximum bending moment in the main span girder, and by more than 500% for the maximum shear force and bending moment in towers. As for stresses, they increase from those obtained for dead loads, by 16% for the maximum cable stress, by 27% for the maximum normal (compression) stress in the main girder, and 158% for the maximum normal (compression) stress in towers.

## OUT-OF-PLANE BEHAVIOUR OF A MODEL BRIDGE

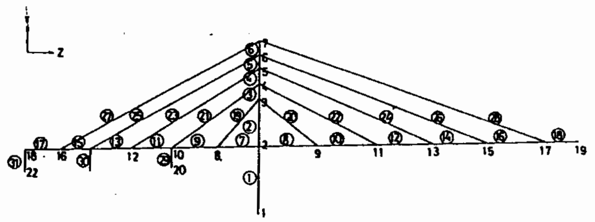
### Structural model, loading condition, and boundary conditions

Here, a case where out-of-plane rotations of piers P3 and P4 take same magnitude at a same time is going to be considered. Hence, the model bridge shown in Fig.4 is remodelled as shown in Fig.7. In the new model, symmetry is taken into account and much less degrees of freedom is introduced, in order to perform the calculation using a micro-computer.

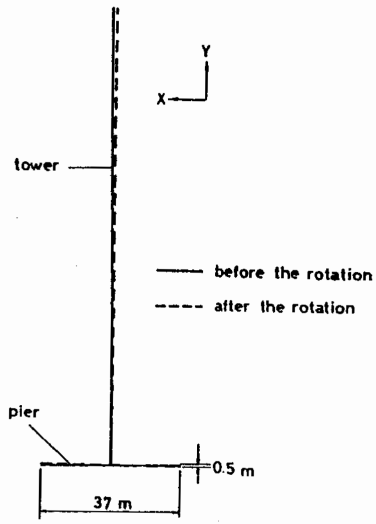
There is no reason to fix the magnitude of settlement to some value. Here, just as a reference value we consider maximum of 0.5m/37m rotation of pier P3 around longitudinal axis (See Fig.7(c)). The length of 37 meters means the width of a bottom cross section of a pier. The boundary conditions and member end conditions are set as shown in Fig.7. In order to avoid numerical difficulty, moments of inertia corresponding bending stiffness and torsional constant of a pendel shoe element are given small values of about 1/100 of the other members' constants.




(a) Structural model used in out-of-plane analyses



(b) Element division (Number of elements is 31 for a half structure)




(c) Out-of-plane rotation of piers P3 and P4

Fig.7 Structural model and element division for out-of-plane analyses

## Deformed configurations, stress resultants and stresses at typical sections

Deformed configurations at each incremental steps are shown in Fig.8. Axial forces in cables, main girders and towers as well as in-plane shear force and bending moment in the latter represent considerable nonlinearity as shown in Fig.9. But these values at 0.5m/37m rotation are only several per cent of those values obtained for dead load action. Whereas out-of-plane shear force, bending moment and torsional moment in main girders and towers change almost linearly with respect to supports settlement.

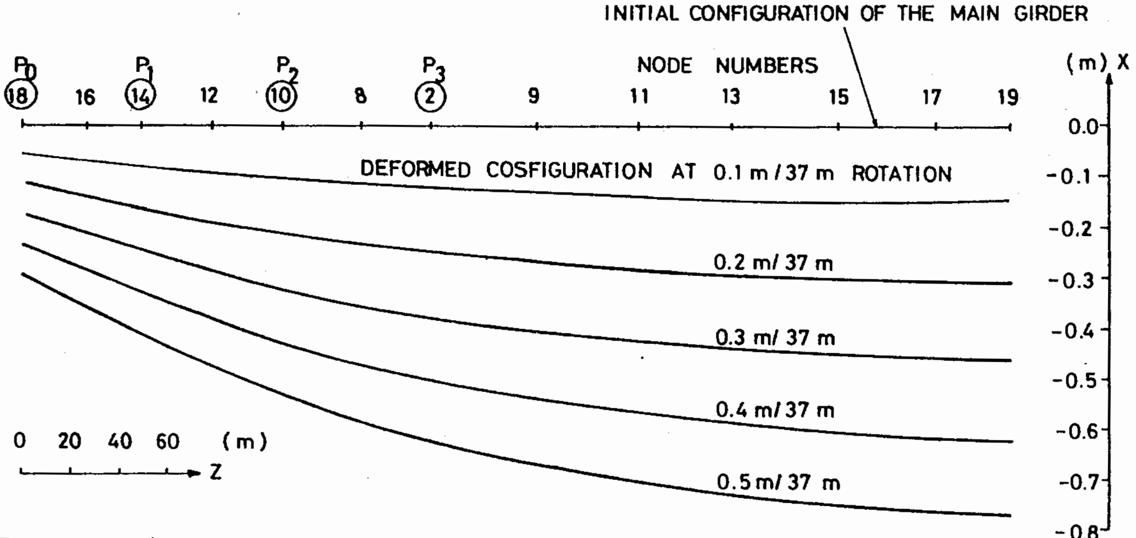



Fig.8 Deformed configurations of the main girders at each incremental steps up to 0.5m/37m rotation of the piers P3 and P4

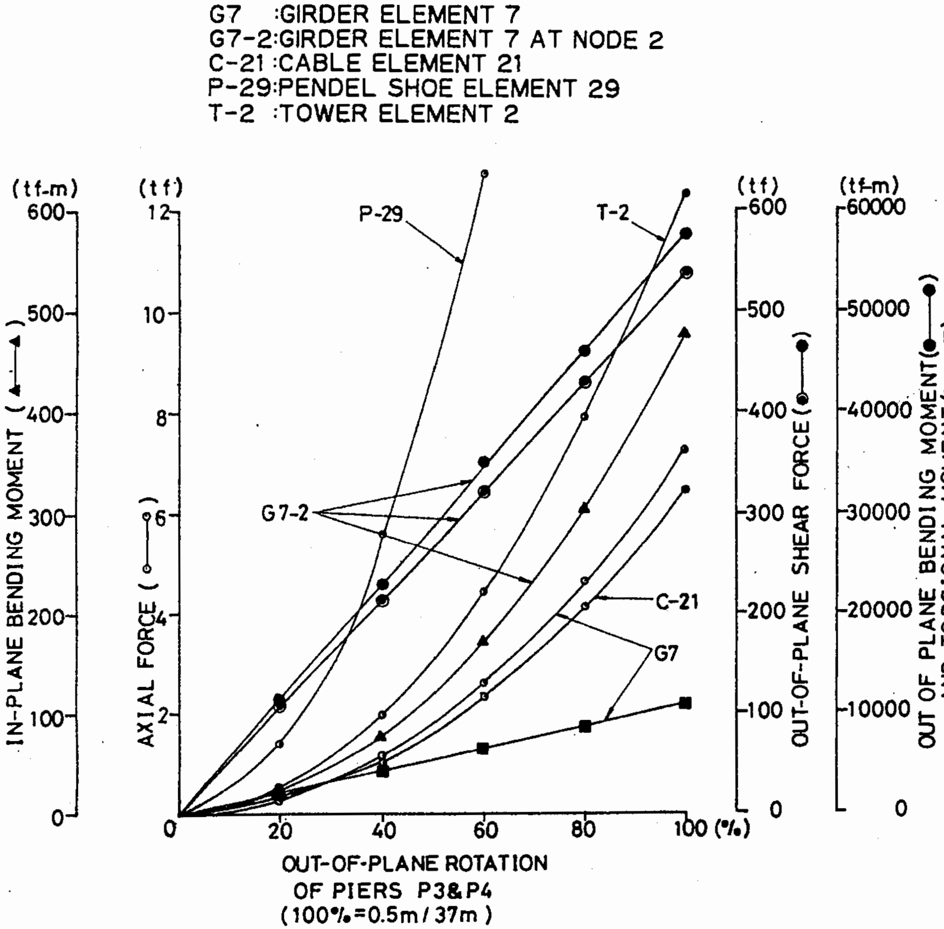



Fig.9 Linearities and Nonlinearities of member forces with respect to the out-of-plane rotations of piers P3 and P4

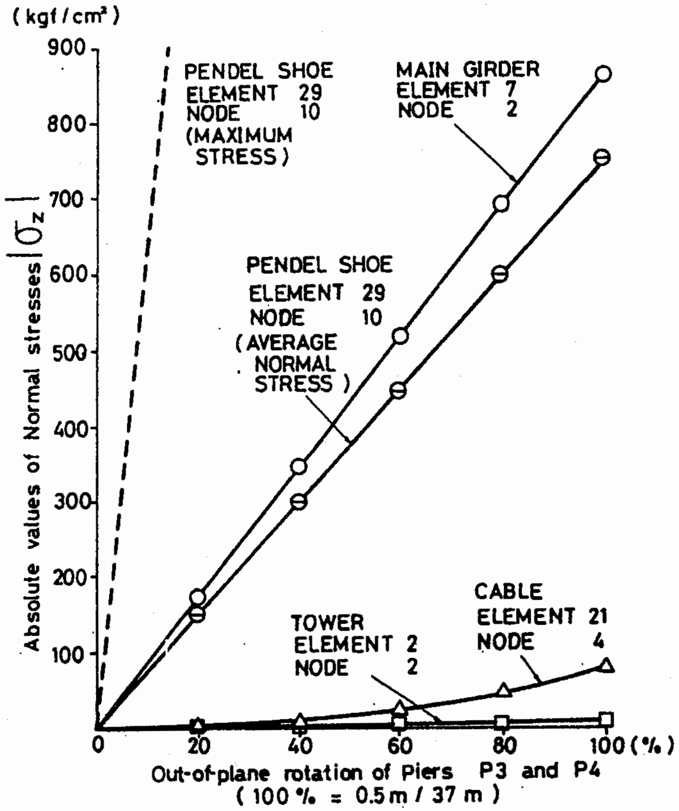
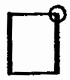
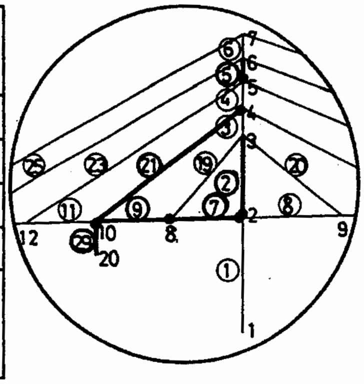



Fig.10 Linearities and Nonlinearities of stresses with respect to the out-of-plane rotations of piers P3 and P4

Table 2 Absolutely largest stresses and their occurring locations in each structural member category for the supports settlement of piers P3 and P4 which causes out-of-plane rotations of magnitude 0.5m/37m

| STRUCTURAL MEMBER CATEGORY / LARGEST STRESS | CABLE | MAIN GIRDER | | TOWER | | PENDEL SHOE |
|---|---|---|---|---|---|---|
| NORMAL STRESS (kgf/cm²) | 7 2 | − 8 7 0 | | 1 7 | | 7 5 0 * |
| SHEAR STRESS (kgf/cm²) | ——— | | − 6 6 0 | | 2 | ——— |
| ELEMENT | 2 1 | 7 | 9 | 2 | 5 | 2 9 |
| NODE | 4 | 2 | 8 | 2 | 5 | 1 0 |
| LOCATION OF THE LARGEST NORMAL STRESS : ○ AND SHEAR STRESS : ▬ |  |  | |  | |  |



* Average stress with respect to the pendel shoe cross section.

At the reference rotation, such sections where large stresses are produced are picked up for each structural category; cable, main girder, tower and pendel shoe. The magnitude of the large stresses and the locations they are developed are illustrated in Table 2. To evaluate stresses, each structural category is considered to have ideal section form as shown in the Table.

## Discussion

It is suggested in Fig.9, that the principle of superposition could not hold any longer in case where there are large out-of-plane rotations of piers. But at least up to about 0.5m/37m rotation, stresses of our concern are considered to be linear as shown in Fig.10, because linear components dominate in this region over nonlinear ones. The only point to have to be taken care of may be that due to rigid connectivity of pendel shoe elements to girders and piers in the transverse plane, excessive large bending stresses are generated as indicated by the dotted line in Fig.10. In the actual structure, this must be avoided by adopting appropriate details. As conclusion, the effects of supports settlements can be summarized as shown in Tables 3 and 4. From these Tables, it can be said that out-of-plane displacements of piers may cause high stresses in main girders, if the displacements are fairly large. For example, out-of-rotations of piers P3 and P4 by 0.5m/37m cause a maximum stress of 870 kgf/cm$^2$ in the main girder which is about 90% of the maximum stress induced by the dead loads in the main girder. In-plane support rotations cause high stresses especially in cables and towers. Thus, careful concerns should be given when single-plane cable-stayed bridges are to be constructed on soft ground so that towers remain as vertical as possible. Nevertheless, within supposable amount of supports settlements in ordinary conditions, additionally generated stresses will not be so large that special countermeasures are needed.

Table 3 Absolutely largest member forces in earth loading case

| MEMBER FORCES / CASE | | STRUCTURAL MODEL : FIG.4 | | STRUCTURAL MODEL : FIG.7 |
|---|---|---|---|---|
| | | DEAD LOADS | P3 PIER IN-PLANE SETTLEMENTS BY 0.5 m (COMPLETE SYSTEM) | P3 & P4 OUT-OF-PLANE ROTATIONS BY 0.5m/37m (PSEUDO-COMPLETE SYSTEM) : ADDITIONAL FORCES |
| CABLE | AXIAL FORCE (tf) | 640 | 690 | 6.5 |
| MAIN GIRDER | AXIAL FORCE (tf)<br>IN-PLANE SHEAR FORCE (tf)<br>IN-PLANE BENDING MOMENT (tf-m) | −4,700<br>520<br>13,000 | −5,200<br>690<br>13,000 | 25<br>13<br>480 |
| | OUT-OF-PLANE SHEAR FORCE (tf)<br>OUT-OF-PLANE BENDING MOMENT (tf-m)<br>TORSIONAL MOMENT (tf-m) | ――<br>――<br>―― | ――<br>――<br>―― | 540<br>51,000<br>11,000 |
| TOWER | AXIAL FORCE (tf)<br>IN-PLANE SHEAR FORCE (tf)<br>IN-PLANE BENDING MOMENT (tf-m) | −5,900<br>160<br>7,700 | −6,300<br>960<br>38,000 | 20<br>22<br>790 |
| | OUT-OF-PLANE SHEAR FORCE (tf)<br>OUT-OF-PLANE BENDING MOMENT (tf-m)<br>TORSIONAL MOMENT (tf-m) | ――<br>――<br>―― | ――<br>――<br>―― | 540<br>35,000<br>43,000 |
| PENDEL SHOE | AXIAL FORCE (tf) | −970 | 970 | 35 |
| | OUT-OF-PLANE SHEAR FORCE (tf)<br>OUT-OF-PLANE BENDING MOMENT (tf-m) | ――<br>―― | ――<br>―― | 290<br>14,000 |

Table 4 Absolutely largest stresses in earth loading case

| STRESS ($kgf/cm^2$) / CASE | | STRUCTURAL MODEL : FIG.4 | | STRUCTURAL MODEL : FIG.7 |
|---|---|---|---|---|
| | | DEAD LOADS | P3 PIER IN-PLANE SETTLEMEMTS BY 0.5 m (COMPLETE SYSTEM) | P3 & P4 OUT-OF-PLANE ROTATIONS BY 0.5m/37m (PSEUDO-COMPLETE SYSTEM) : ADDITIONAL FORCES |
| CABLE | NORMAL STRESS | 2,600 | 3,100 | 72 |
| MAIN GIRDER | NORMAL STRESS | −1,000 | −1,300 | −870 |
| | SHEAR STRESS | 200 | 270 | −660 |
| TOWER | NORMAL STRESS | −1,000 | −2,700 | 17 |
| | SHEAR STRESS | 60 | 200 | 2 |
| PENDEL SHOE | NORMAL STRESS | −320 | 320 | 750* |

* Average stress with respect to the pendel shoe cross section.

## CONCLUSION

A simplified method for the analysis of single-plane fan-type cable-stayed bridges, which is going to be constructed on soft clay and expected to be subjected to nonuniform supports settlements, has been presented. The method is composed of two parts. One is geometrical nonlinear analysis based on the method developed in [1]-[3]. Here, a pair of pendel shoes is represented by a newly introduced frame element and the whole structure is modelled by this and ordinary frame elements. The other is stress evaluation according to thin walled members theory. In the method, the concept of pseudo-complete system is adopted, which enables us to delete complex simulation of construction stages.

The method has been applied to the analysis of Chao-Phya like bridge subjected to supports settlements which induce out-of-plane rotations of piers, and has shown the possibility of unified analysis for both in-plane and out-of-plane behaviours. The calculated results indicated that for this type of a bridge, supposable amount of supports settlements will not cause considerable stresses in the superstructure.

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# ANALYSIS AND OPTIMAL PRETENSIONED FORCES FOR CABLE SUSPENDED STRUCTURES

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## SUMMARY

A general purpose software for static and dynamic analyses of cable structures is developed. An optimization subroutine for evaluating the optimal values of the pretensioned forces in view of minimizing the maximum nodal deflections is incorporated. The governing equation of motion is formulated based on the discrete element approach. Direct time integration is carried out using Newmark's $\alpha$-method and the optimal values of pretensioned forces obtained by the simplex method of Nelder and Mead.

## INTRODUCTION

Cable suspended roof structures permit a large area to be covered with only a relatively small amount of structural materials. In recent times, these structures have attracted the attention of architects and engineers because of their combined lightness, strength, economics and the elegance of their forms. Several types of suspended roof structures have been developed, for example the plane prestressed cable trusses, prestressed cable networks, and various types of cyclically symmetric, circular cable structures.

The problem of analyzing cable systems is rather complex since cable structures, being inherently flexible, exhibit a highly nonlinear behaviour when loaded. The analysis must therefore include finite deformations and the effects of internal forces. Consequently, most of the methods of analysis proposed are based on some iterative schemes of which the Newton-Raphson method is one of the most commonly used. It can be said that after much research efforts [1-5] since 1950's, the problem of analyzing cable structure for static effects has been adequately met. However the dynamic analysis of