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Load-Deflection Characteristics of Tubular Steel Towers

Caractéristiques charge-déformation de pylônes tubulaires en acier

Last- und Durchbiegungs-Eigenschaften von Stahlrohrmasten

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SUMMARY
A design concept for tubular steel electrical transmission towers which are required to be earthquake and wind resistant is described. Experimental and non-linear analyses are performed to investigate the load-deflection characteristics and ultimate strength of the towers, designed in accordance with this concept.

RESUME
Une conception de pylônes tubulaires en acier pour lignes à haute tension devant résister aux séismes et au vent est présentée. Le résultat de l'expérience et l'analyse non-linéaire sont décrits, le but étant de déterminer les caractéristiques charge-déformation et la résistance maximale de l'ouvrage, qui a été réalisée conformément à cette conception.

ZUSAMMENFASSUNG
Ein Entwurfskonzept für Hochspannungsmasten aus Stahlrohren, die gegen Erdbeben und Wind widerstandsfähig sein sollen, wird hier dargestellt. Der Versuch und die nichtlineare Analyse werden beschrieben, um die Last- und Durchbiegungs-Eigenschaften und die Maximalstärke der Masten, die nach diesem Konzept entworfen wurden, zu untersuchen.

1. INTRODUCTION

An electrical power demand is increasing in Japan and the construction of ultra high voltage electrical transmission lines for power supply to big city areas is needed. Self-supporting square towers of double warren trusses are adopted in Japan as the towers of transmission lines, and such ultra high voltage transmission towers will be rigidly jointed trusses using tubular steel members, as shown in Fig. 1. It is very important to keep the integrity of transmission facilities, considering their social influence. In discussing the safety of structures in Japan, the earthquake and wind resistant design is the most important factor. It is necessary to define the load-deflection characteristics and ultimate strength of structures, because the deformation capacities of structures play an important role in the earthquake and wind resistant design.

This study consists of the following items.
- Design philosophy considering the deformation capacities of towers.
- Experiments for determination of load-deflection characteristics up to ultimate state and ultimate strength of towers, designed according to above item.
- Development of a nonlinear analytical method which can predict the characteristics indicated in above item.

2. DESIGN PHILOSOPHY

The slenderness ratios of the tubular members of ultra high voltage electrical transmission towers are expected to be in the range shown in Table 1. The collapse process of the body, the arm and the leg should be such that, assuming the distance between centers of nodes as the buckling length, the bracing members do not buckle before the column members effectively display their strength and deformation capacity in individual buckling. When buckling of column members which have considerably small slenderness ratios as shown in Table 1 precedes, the tower is expected to display deformation capacities because of the column members displaying sufficient stress-carrying capacities with plastic deformations.

Deformation capacity of the whole tower will be increased, if stresses in all members of the tower are uniformly distributed and all panels yield simultaneously. We suggest that the arm should be designed such that it fails earlier. This will contribute to the prevention of a failure of the body, thereby lessening damages and shortening the schedule of restoration works.

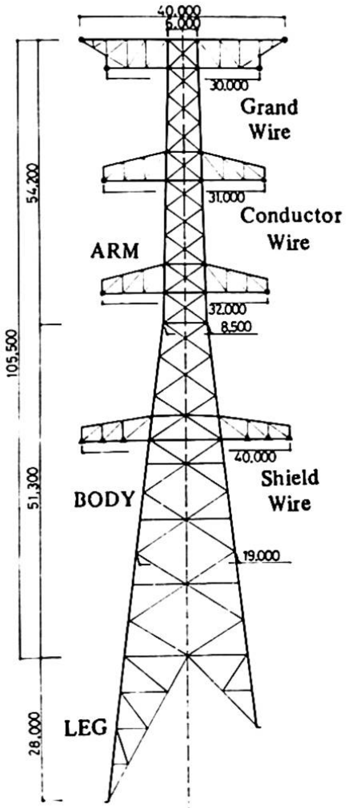


Fig. 1 Example of ultra high voltage electrical transmission towers (mm)

Table 1 Slenderness ratios of members

		Column member	Bracing member
Body	Upper from changed point of inclined angle of columns	30 ~ 60	60 ~ 80
	Lower from changed point of inclined angle of columns	20 ~ 40	80 ~ 150
Arm		30 ~ 40	40 ~ 100
Leg		10 ~ 40	80 ~ 150

3. NONLINEAR ANALYSIS

This analytical method is based on the finite deformation theory derived from the principle of the virtual work. The equilibrium equations of members are formulated for a finite element model in three dimensional stress condition. Regarding heavily deforming or yielding members, we can get higher accuracy of the analysis by using inner nodes in the members.

The relationship of normal stress σ and normal strain ε in materials is determined such as to satisfy the requirement of stub column tests, as shown at M/My=0 in Fig. 2.
Full plastic moment-axial force interaction curve of tubular members is shown in Fig. 3. As for the members subjected to axial forces and bending moments, the relationships of σ and ε are determined like Fig. 2 by finding the full plastic values for normal stress from Fig. 3 and moving the relationship of σ and ε at M/My=0 parallel after yielding, as indicated in Fig. 2.

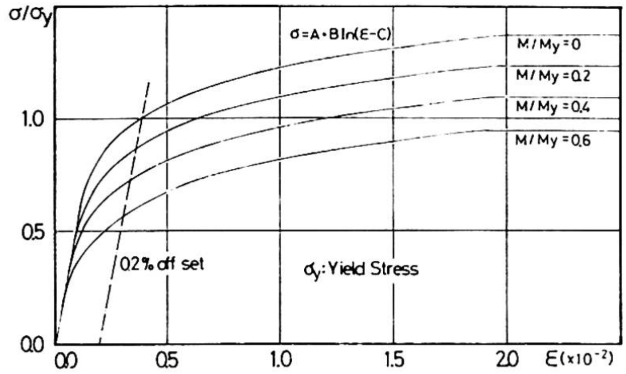


Fig. 2 Stress-strain relationships

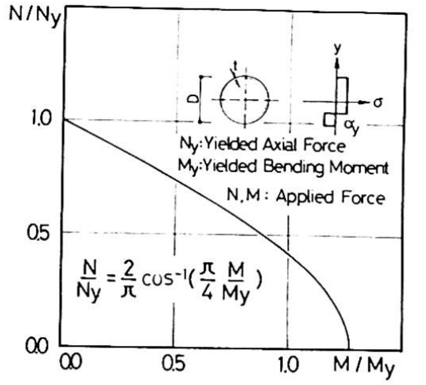


Fig. 3 Moment-axial force interaction curve

4. LOAD-DEFLECTION CHARACTERISTICS AND ULTIMATE STRENGTH OF THE TOWER

4.1 Body

We carried out model experiments on 4 kinds of the body shown in Fig. 4.
The specimens consist of the following factors.

- The specimens are trepaned from the neighborhood of the changed point of the inclined angle of the columns and at the top of the specimens, horizontal forces are caused to act.
- The specimens A and B have different inclined angles of the columns.
  The specimens C and D have the same shape, but different directions of applied forces.
- All the specimens are designed such that buckling of column members precedes that of bracing members. The slenderness ratios of buckled column members are 15 in A and B and 35 in C and D, if the distance between centers of nodes is assuming as the buckling length.
- Column members are continuous and bracing members are connected with column members through gusset plates.

The relationships of load-deflection to force direction given by this experiment and nonlinear analysis are shown in Fig. 5, in which loads given by the conventional analysis, that is to say, the linear static truss analysis with pin-joints of nodes are shown.

The following can be said from the results of the experiment and the analysis.
The yielding load P'y by the nonlinear analysis considering the effect of bending moments corresponds to the load under which the stiffness of the elastic region in the experimental result begins to lower, and the yielding load Py by the

conventional analysis considering axial forces only corresponds to the load under which the stiffness in the experimental result begins to lower rapidly. The ultimate strength P'u by the nonlinear analysis nearly corresponds to the experimental result, but the ultimate strength Pu by the conventional analysis is a little lower than the experimental result. The ductility factors, i.e., the ratios of the deflection at the ultimate strength in the experiment to the deflection at the yielding load by the conventional analysis, are about 3.5 in the specimen A, 2.5 in B and C, and 3.0 in D. The result of the non-linear analysis developed in this study agrees with the experimental result and the availability of this analytical method has been proved.

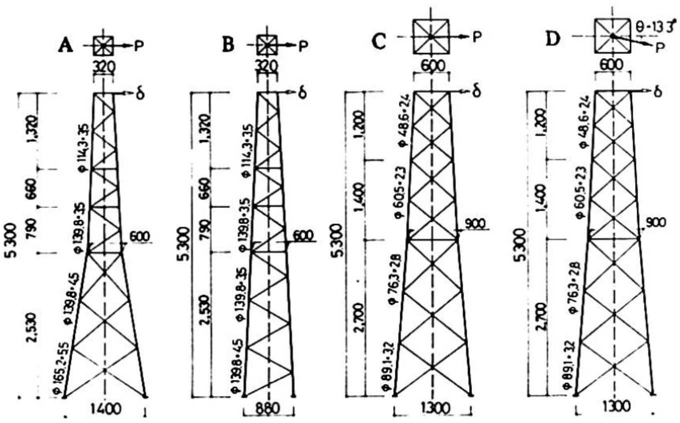
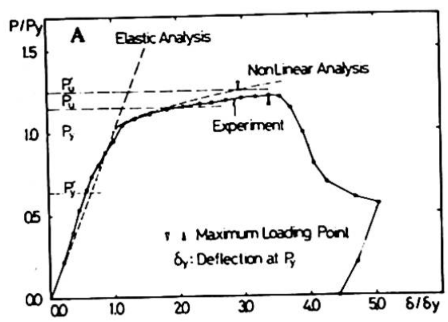


Fig. 4 Specimen of body (mm)



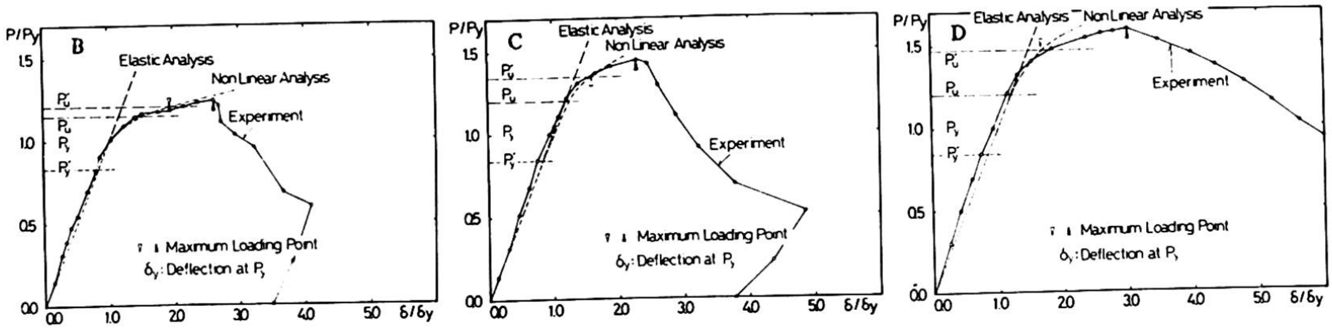


Fig. 5 Load-deflection relationships in body

4.2 Arm

Regarding 3 kinds of the arm shown in Fig. 6, full scale field experiments were performed. The relationships of load-deflection given by the experiment and the nonlinear analysis in the specimen C are shown in Fig. 7. We can obtain almost the same result on the arm as on the body.

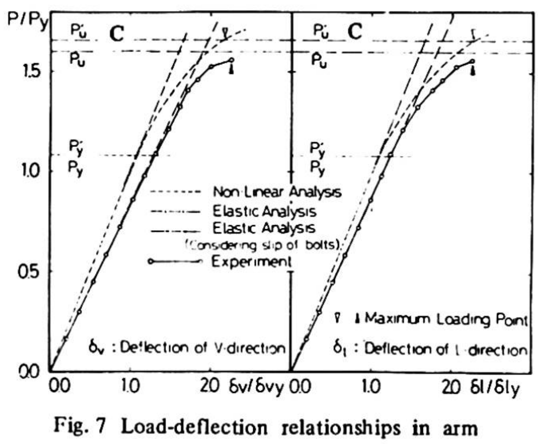


Fig. 7 Load-deflection relationships in arm

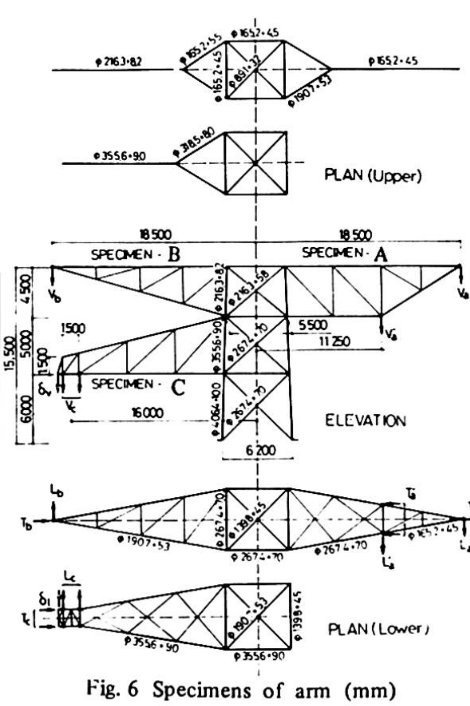


Fig. 6 Specimens of arm (mm)

4.3 Leg

Model experiments were performed on 8 kinds of the leg shown in Fig. 8 and Table 2. The specimens consist of the following factors.

- Variable factors of the specimens are slenderness ratios of column members, number of panels and strength ratios of column and bracing members, as shown in Table 2.
- The ratio of applied forces between column members and bracing members is 10.
- All the specimens are designed such that yielding and buckling of column members precede those of bracing members, except the specimen G.
- Column and bracing members are continuous and reinforcement members are connected with them through gusset plates.

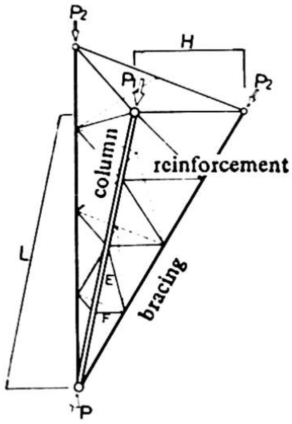


Fig. 8 Specimens of leg

Prediction of buckling load of the leg

The relationships of load-deflection are found by the nonlinear analysis described in chapter 3. Regarding

Table 2 Specimens and results of experiment and analysis

Specimens	Slenderness ratios of column members	Numbers of panels	Ratios of allowable loads: column / bracing	e-member / column	f-member / column	Buckling load: experiment / theoretical	Failed member
A	20	2	6.3	0.07	0.06	0.92	column
B	20	3	6.0	0.07	0.07	0.90	column
C	20	4	5.9	0.13	0.07	1.03	column
D	20	4	4.9	0.13	0.09	0.99	column
E	38	2	7.6	0.10	0.17	0.96	column
F	38	3	6.1	0.12	0.20	0.91	column
G	38	4	11.3	0.13	0.21	0.97	bracing
H	38	4	5.7	0.13	0.21	0.92	column

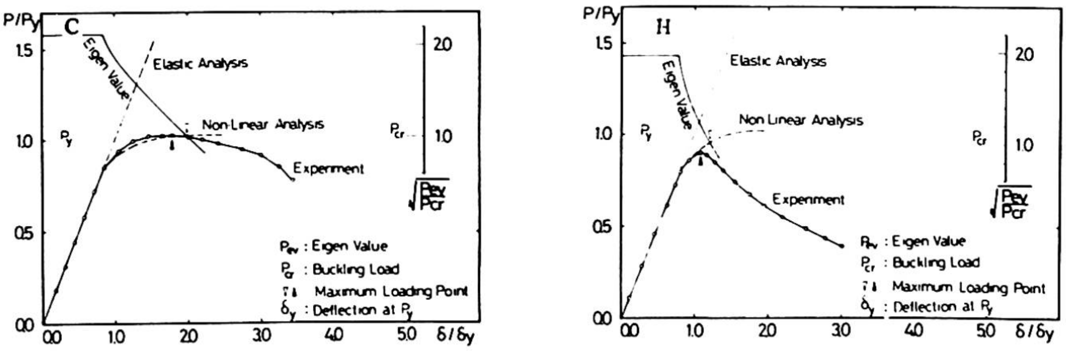


Fig. 9 Prediction of buckling load of leg

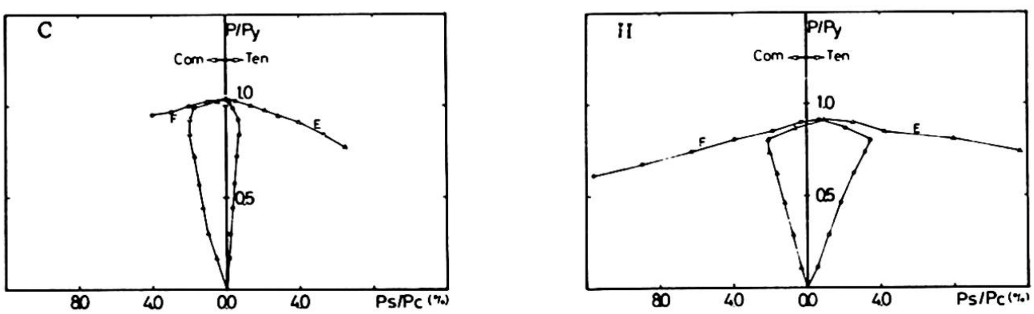


Fig. 10 P/Py–Ps/Pc relationships

each load step in the nonlinear analysis, the eigen values are obtained by performing an eigen value analysis considering material and geometrical nonlinearities. In this case, as shown in Fig. 9, we define the load under which those curves intersect as the buckling load. Comparison between the analytical result and the experimental result is made in Table 2. We can estimate the buckling load accurately by using this analytical method.

Necessary strength of bracing and reinforcement members for individual buckling of column members

For the leg design, a method is available in which bracing and reinforcement members are designed as supporting members of the columns. In this method, it is a problem to decide the necessary strength of the supporting members for individual buckling of the column members to be caused firstly in the leg. As regards the specimens C, H, in which column members suffered individual buckling, the ratios of axial forces of the column members Pc to the axial forces of the reinforcement members Ps are shown in Fig. 10. In this experiment, it is obvious that when the strength of bracing members is appropriately overestimated and the strength of the reinforcement members is designed to be about 7% of the strength of the column members, the leg will be failed by individual buckling of the column members.

5. CONCLUSION

Load-deflection characteristics and ultimate strength of tubular steel rigid-jointed truss towers with small slenderness ratios of column members are quantitatively determined in this study. These characteristics will be useful in the earthquake and wind resistant design of electrical transmission line towers.

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