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Autor: Sera, Kohsaku / Konomi, Shinzo / Maehara, Satoshi
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Design of Steel Shell Frames of the Fukuoka Exhibition Hall

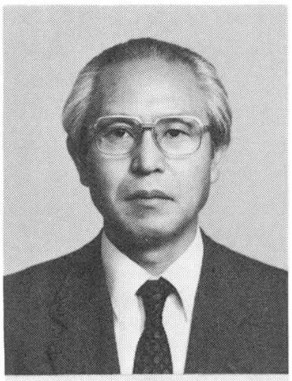
Projet de la couverture en acier de la salle d'exposition de Fukuoka

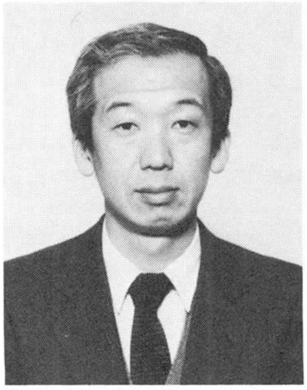
Entwurf der Schalenstahlrahmen der Fukuoka-Ausstellungshalle

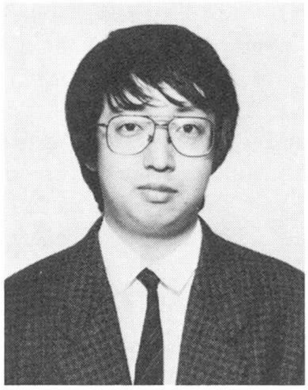
Kohsaku SERA
Director
Nihon Sekkei, Inc.
Tokyo, Japan

Shinzo KONOMI
Senior Structural Eng.
Nihon Sekkei, Inc.
Tokyo, Japan

Satoshi MAEHARA
Structural Eng.
Nihon Sekkei, Inc.
Tokyo, Japan







SUMMARY

The roof structure of the Fukuoka Exhibition Hall Building is composed of large scale steel vaulted shells. Prestressing is used to reduce vertical displacements. The design method for metal prestressing shell structures and the tests to check the safety of the bearing plates that fix the prestressing wire and, to confirm the relaxation are being presented in this paper.

RÉSUMÉ

La structure porteuse de la toiture de la salle d'exposition de Fukuoka se compose d'éléments de voiles cylindriques en acier afin de réduire les déplacements verticaux. La communication traite des données de base servant à la conception des structures porteuses de coques métalliques ainsi que des essais de charge ultime des plaques d'ancrage et de la relaxation des fils de précontrainte.

ZUSAMMENFASSUNG

Die Dachkonstruktion des Fukuoka-Ausstellungsbaus besteht aus grossformatigen Stahl-Tonnenschalen, die zur Verminderung der Durchbiegung vorgespannt werden. Der Beitrag behandelt die Entwurfsgrundlagen für vorgespannte Stahlschalentragwerke sowie die Versuche zur Sicherheit der Ankerplatten und der Relaxation der Spanndrähte.

1. INTRODUCTION

The architectural design of the roof was undertaken by imagining sea waves, because the structure is located in front of the sea in Fukuoka city (see Fig. 1). The Fukuoka Exhibition Hall Building consists of 4 stories (two basement floors) with a total architectural area of 40,581 m^2, a maximum height of 31 m and seating capacity for 15,000 people. This long span building will be mainly used as an exhibition hall; there is also a small gymnasium and council room. The dimensions of the roof structure are 144 x 120 m and the rise of the vault is 7.5 m. This roof is composed of three continuous steel latticed vaults and two steel shell wings arranged at both sides of the vaults as shown in Fig. 2 & 3. The roof structure is supported by four big columns, whose spans are 36 m and 100.8 m, with other smaller columns at the shell wings. The steel latticed vaults and wings are composed of welded built-up H-steel sections (BH- 850 x 200 mm) arranged in the longitudinal direction, and steel trusses (H = 850 mm) in the transverse direction as shown in Fig. 4. The steel latticed vault is reinforced by steel ribs arranged every 14.4 m in the transverse direction in order to avoid buckling, general instability and to reinforce against non-symmetrical loads such as seismic and wind loads. Additionally, these ribs are utilized as maintenance routes and for lighting and mechanical uses. Instability behavior and the stiffening effect of ribs, together with the dynamic behavior under vertical and horizontal earthquake vibrations, and prestressing effects on displacement and stress, have been analyzed in order to design this beautiful shell structure. Simple connections are used and wind tunnel tests were performed to determine the wind force coefficient.

2. STRUCTURAL ANALYSIS

2.1 Design Load

The design live load of the roof is 980 N/m^2 which allows for the suspension of various exhibits, and also the possibility of having concentrated loads of 20 kN at the frame nodes. Wind load is determined using the results of the wind tunnel test. The model scale is 1/300, and roughness of the surrounding condition of the site is considered. Finally, 0.9 to 5.5 kN/m^2 of wind load is adopted allowing for the effects of gust and a 150 years return period. As for the aseismic design, vertical and horizontal loads on the roof structure are considered. Vertical and horizontal seismic load coefficients are determined by numerical dynamic analysis in time domain, using three actual earthquake records. The first natural period in the vertical direction of the roof is 0.71 sec. and the first natural period in the horizontal direction, of the whole structure including the main frame that supports the roof structure, is 0.33 sec.

2.2 Stress Analysis

Seven cases of structural frame analysis have been undertaken to design the roof structure as shown in Table 1. There are two analytical models; one considers the whole model with some secondary members thinned out and the second, considers a quarter of the structure taking into account all members and the symmetry conditions. Combinations of the stresses used to design the section are shown in Table 2. Boundary (support) conditions are set considering the order of the construction. The support condition against self weight of the steel roof structure is initially considered as roller (free) in the long span direction in order to avoid large internal forces in the supporting columns due to horizontal movements of the roof, when the temporary supports used for erection are removed. The support conditions against other loads such as live loads, seismic loads, etc. are considered as spring supports. The results are shown in Table 3, where the vertical deflection at the center of the roof and horizontal displacement at the supported point of the roof are presented. The design of the sections of the members is performed considering a combination of axial force and two directional bending moments.
One example of the analysis results is shown in Fig. 5.

3. PRESTRESSING

It is too difficult to pre-camber this roof structure, therefore, prestressing is used in order to get the least possible deflection. Details of the prestressing are shown in Fig. 6. Unbonded tendons to avoid rust are installed in the steel pipe at the upper and lower chords of the edge beam in the bottom of the steel vault. The tensile forces in these prestressing cables are restricted by the following equation for any load case.

$s_0 \leq 0.6$ Pu and 0.7 Py
$s_{max} \leq 0.7$ Pu and 0.85 Py

where

s_0 : Stress by prestressing
s_{max}: Stress by $s_0 + s_\wedge$
$s_\wedge$: Stress due to seismic load, thermal stress, live load, etc.
Pu : Ultimate tensile strength of tendon
Py : Yield tensile strength of tendon

Stresses due to prestressing are shown in Fig. 7, and the deflection of the center of the roof is +7.2 cm. The maximum tensile forces in the prestressing cables are obtained considering the worst load case condition and are equal to 0.67 Pu and 0.79 Py. The anchoring plates for these prestressing cables are analyzed using the Finite Element Method. These anchoring plates are designed such that the maximum stress does not go beyond 0.75 Fy; where Fy is the yielding strength of the anchoring plate. Thus the plate thickness for a prestressing force of 3000 kN is 90 mm, and for the case of 1500 kN is 70 mm. Relaxation of the total system and stress of the anchoring plate have been confirmed by experimental tests of the actual size test piece shown in Fig. 8. The maximum relaxation during 100 days of the total system composed unbonded tendon, anchoring plate and steel pipe was 2.6 %, as shown in Fig. 9. Distributions of the strain at the surface of anchoring plate are shown in Fig. 10.

4. CONCLUSIONS

The roof structure of the Fukuoka Exhibition Hall Building is composed of large scale steel vaulted shells. Prestressing is used to reduce vertical deflection. Studies investigating the effect of the rib reinforcement to increase the stability behavior; the effect of prestressing to reduce vertical deflections and, the experimental tests of the prestressing system were performed in order to design the large-span thin steel vault structure reported in this paper. And the design method for metal prestressing shell structures, the test methods to check the safety of the anchoring plate that holds the prestressing wire and to confirm the relaxation are also presented in this paper.

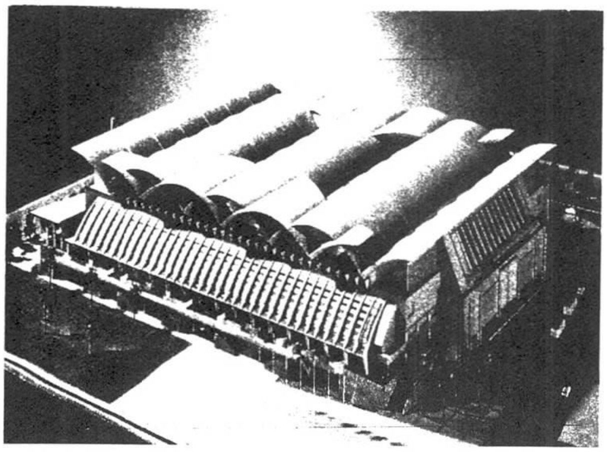


Fig. 1 Fukuoka Exhibition Hall

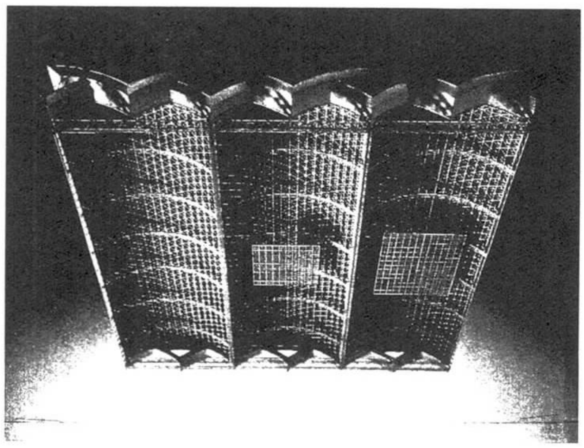


Fig. 2 Photo of the roof framing

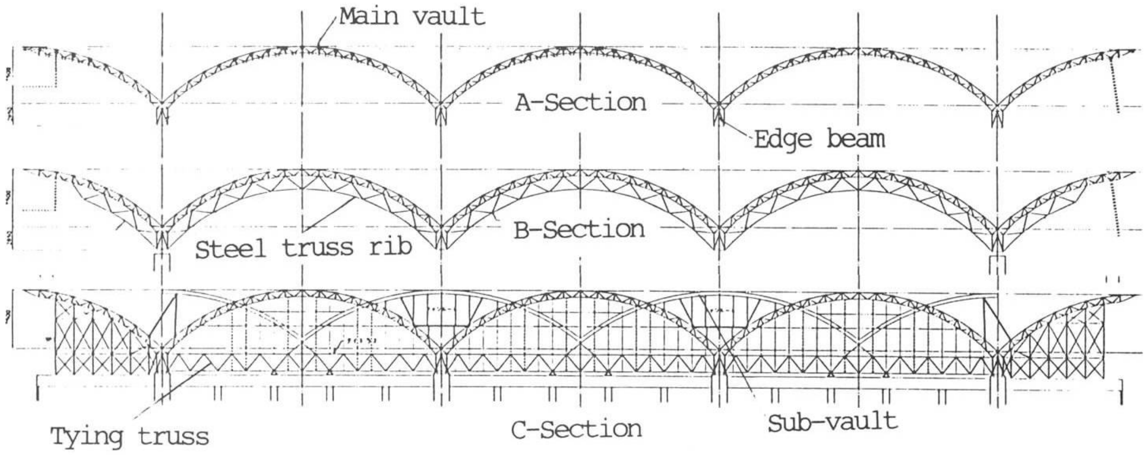


Fig. 3 Elevation of the roof framing

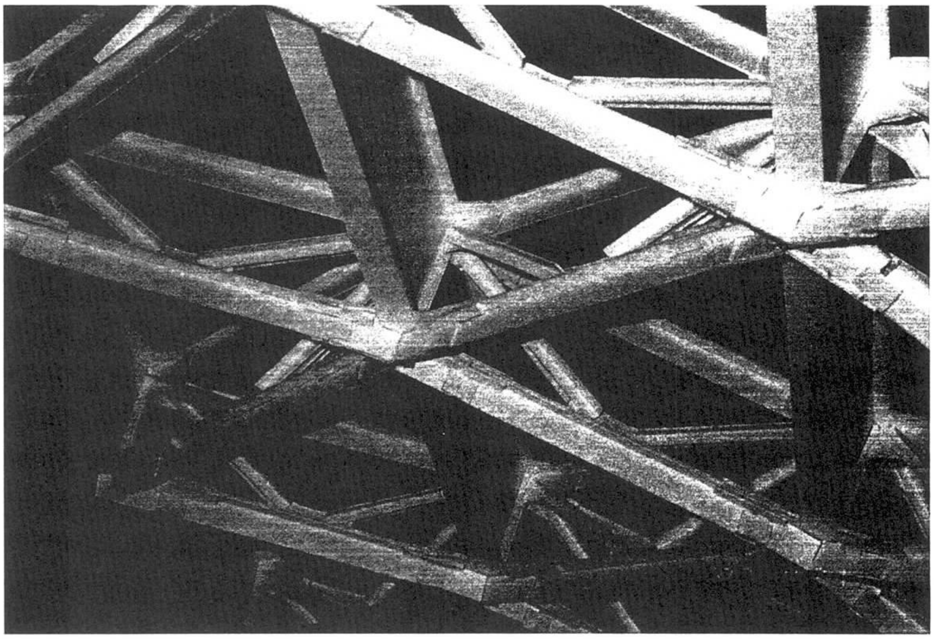


Fig. 4 Photo of the main shell vault

Table 1 External load cases

No.	Load case
1	Vertical load (Steel weight only)
2	Vertical load (Live load etc.)
3	Thermal stress (± 15℃)
4	Thermal stress (± 30℃)
5	Earthquake
6	Wind forces
7	Prestressing

Table 2 Stress combinations for designing section

No.	Combination	Allowable stress
1	1+2+3	long term
2	1+2+3+7	long term
3	1+2+4	short term
4	1+2+3+5	short term
5	1+2+3+5+7	short term

Table 3 Vertical deflection at the center of the roof

No.	Vertical deflection (cm)*1	Horizontal movement (cm)*2
1	− 8.6	
2	− 17.0	2.25
3	± 0.63	± 0.66
4	± 1.23	± 1.31
5	0	1.73
6	+ 8.74	
7	+ 7.2	

*1 − : downward, +:upward
*2 longitudinal direction (supported point)

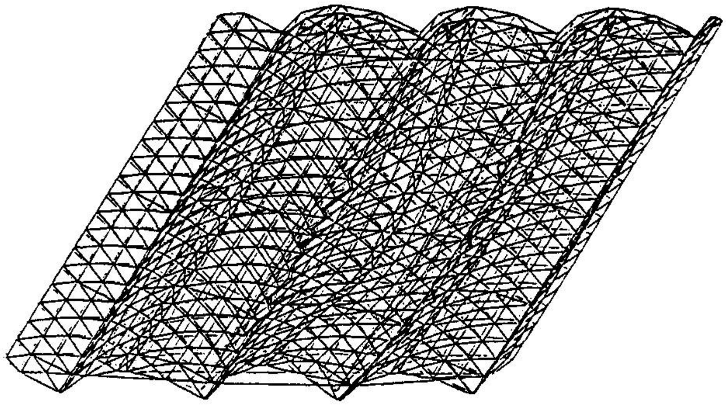


Fig. 5 Results of the analysis

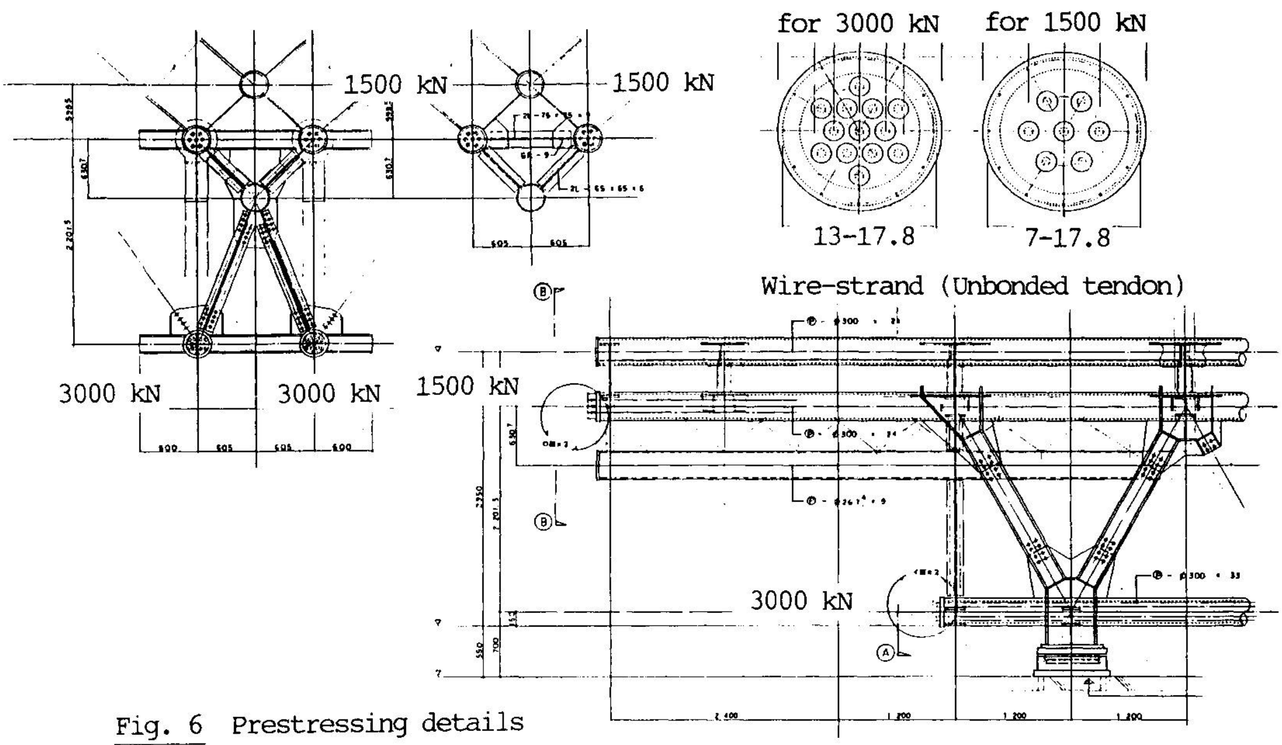


Fig. 6 Prestressing details

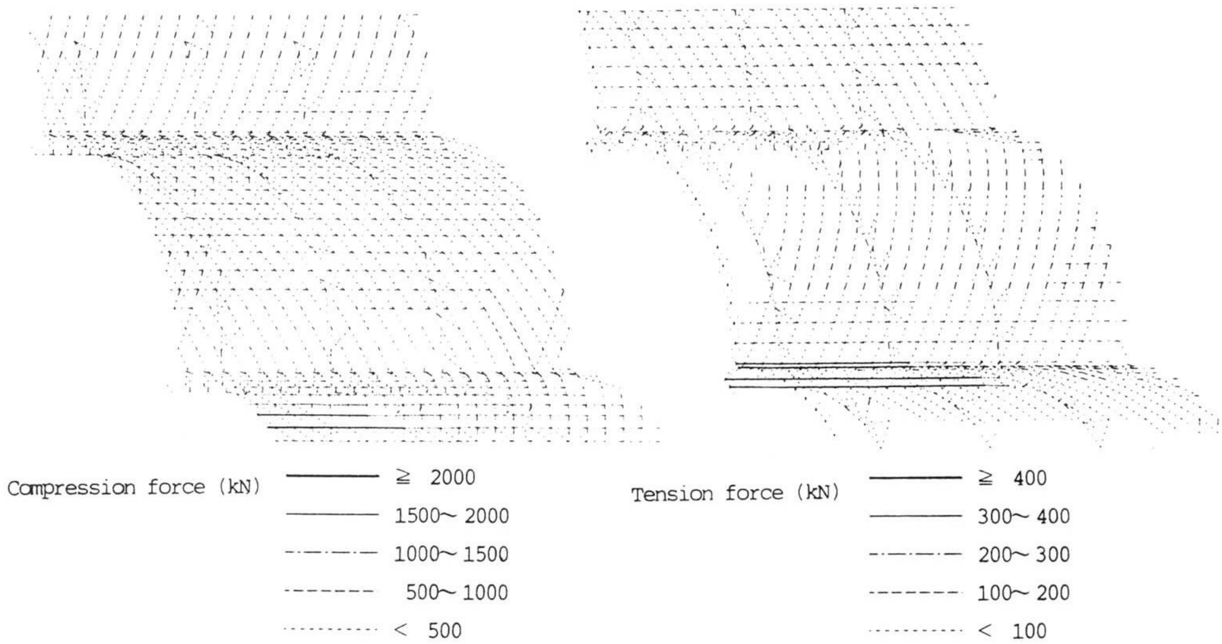


Fig. 7 Frame stresses against prestressing

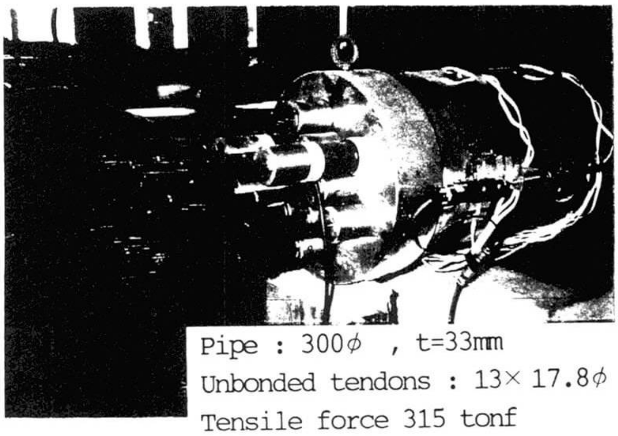


Fig. 8 Test sample

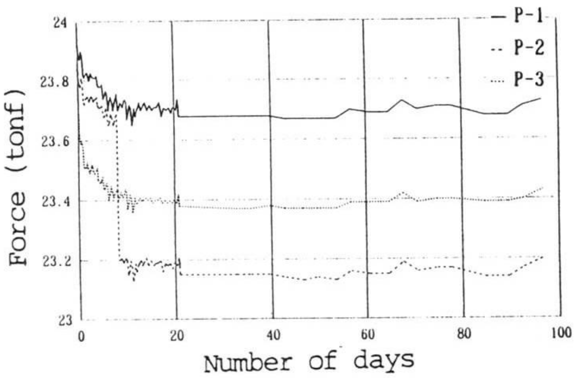


Fig. 9 PC Post-tension time-story

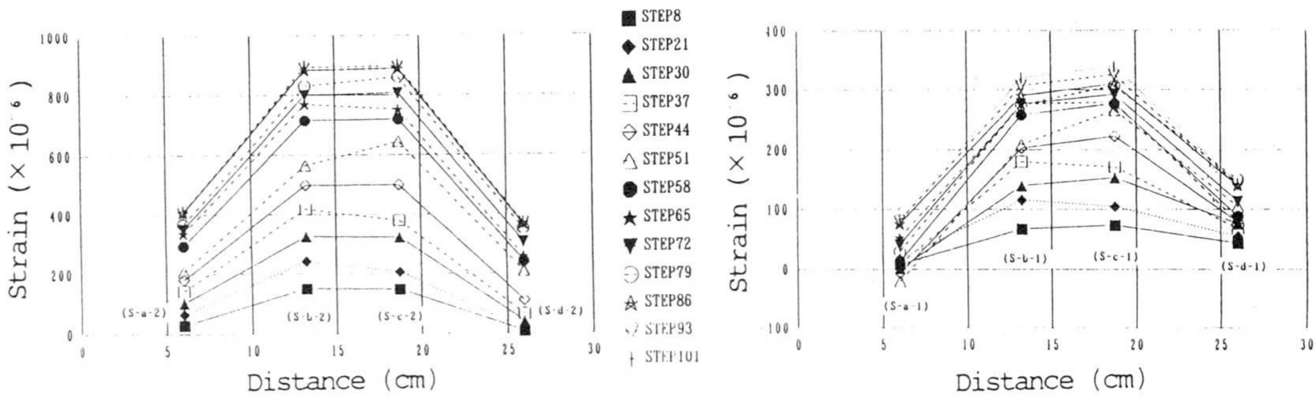


Fig. 10 Anchor plate strain when Post-tensioning