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Development of Prestressed Steel Truss "Super Wing"

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The body can be viewed from the next page.

Development of Prestressed Steel Truss "Super Wing"*


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Synopsis:

Kawasaki Steel Corp. and Shimizu Corp. have developed a unique long span structural system called ***Super Wing****, which utilizes a prestressed steel truss (PSST) and a sliding construction method. During the development of Super Wing, a full-scale 100-meter-span model was tested, and the structural behavior of the PSST was observed both during the prestressing stage and execution of the sliding construction. In addition, the relaxation of prestressing strands was studied experimentally. The wind pressure coefficient of this long span structure was also investigated using a wind tunnel. As a result of these test, Super Wing was proven to be successful as a structural design concept and construction method.*

1 Introduction

Long span structures have come of age as a state-of-the-art construction technology. Large open space unobstructed by columns allows easy viewing of sports and exhibitions and makes for more efficient operations in manufacturing plants, distribution warehouses, etc. and thus offer distinct advantages to the owner. A structural system which is flexible enough to meet a variety of needs and also provides flexible, muti-purpose space has been sought for many years.

In connection with the construction of weather-proof facilities and warehouses for flexible use in our own Chiba Works integrated steel plant and the renovation of existing production facilities, Kawasaki Steel Corp. (KSC) began developing long-span structures in cooperation with Shimizu Corp.

In 1985, development of the Super Wing, a unique, sophisticated method unprecedented in the world was completed. The new design was developed using a full-scale model (**Photo 1**), and the actual roof construction for the NHK Spring Co., Ltd.'s new plant in Yokohama and the event hall for the Seto-Ohashi (new Inland Sea bridge complex) exhibition. These installations have proven successful in all respects.

This paper provides detailed descriptions of the technical features, significant parameters, and criteria relative to the design and erection using the Super Wing system.

2 Features of Super Wing

2.1 Prestressed Steel Truss (PSST)

2.1.1 Structural principles

The main structural feature of Super Wing is the PSST, in which the lower chord members of a steel truss are prestressed. In **Fig. 1**, a truss deformed due to dead load is indicated by the solid lines. Prestressing strands installed along the lower chord members are tensioned and fixed at both ends, thus introducing compressive-stress in these members, which results in uplift on the truss.

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Photo 1 Actual experimental scale model

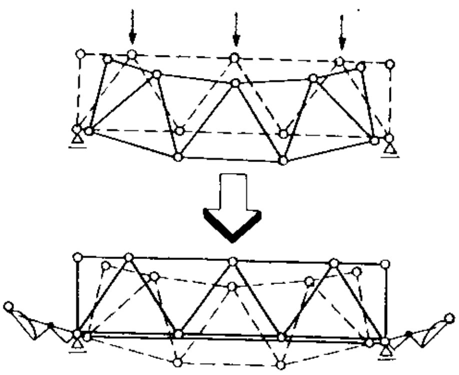

Fig. 1 Structural principle

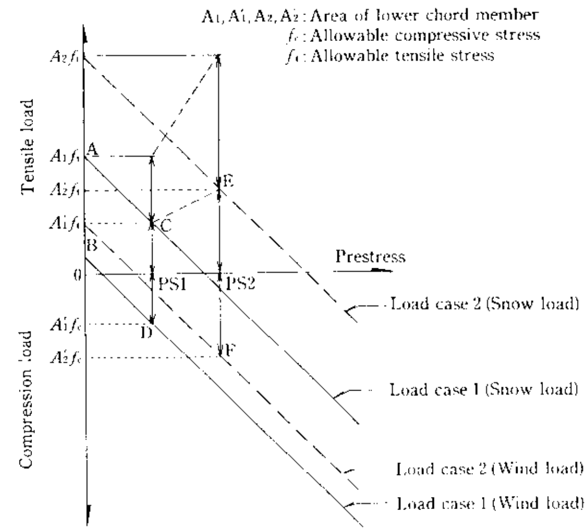

Fig. 2 Axial load of lower chord due to prestress

2.1.2 Advantages of PSST

(1) Reduction in Steel Weight

Figure 2 shows schematically the relationship between the axial load acting on the lower chord and the amount of prestress. Tensile and compression loads on a conventional truss are shown along a longitudinal axis at zero prestress. For loading case 1, which is shown by a solid line in Fig. 2, the cross section of the lower chord member of a conventional truss is required to sustain the tensile load ($A_1 \cdot f_t$) under snow load (point A). With PSST, the lower chord axial load decreases to the right and down as shown in the figure as the amount of prestress increases. If a prestress amount PS1 is selected so that a lower chord section A_1' is fully effective within the allowable stress for both tensile load

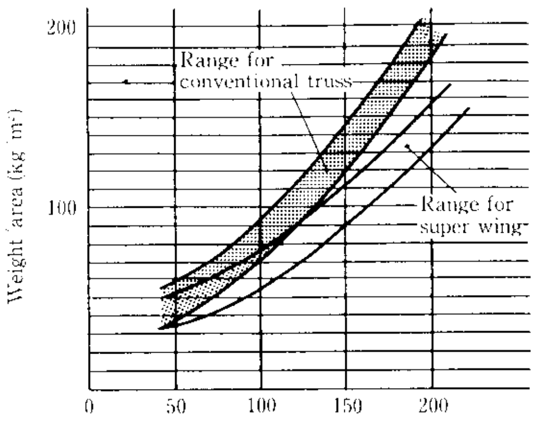

Fig. 3 Typical weight comparison of roof structure

under snow load and compression load under wind load, then the cross sectional area of the lower chord member can be minimized, resulting in a reduction of steel weight.

(2) Advantage for Heavy Snow Load and for Long Spans
The broken lines shown in Fig. 2 assume a large tensile stress as in areas with much snow, although similar conditions are created by the weight of roof material in long span structures. Points E and F indicate the load points for these cases; the required section of lower chord member for PSST becomes A_2', reduced from A_2 (the required section for a conventional truss), resulting in a larger reduction ratio than that for case 1. For PSST, the greater the volume of snow or the longer the span, the more the steel weight is reduced. The relationship between spans and the weight of roof steel material is shown in **Fig. 3**.

(3) Reduction in Truss Depth
As mentioned above, the steel weight can be reduced using PSST compared with a conventional truss; the depth of truss can then be made smaller, resulting in a flat structure with a long span, something impossible using conventional method.

(4) Simple Extension
The basic construction of PSST consists of repeated installation of simple trusses, facilitating extension work laterally in the direction of the trusses. In addition, intermediate columns are unnecessary, making possible efficient utilization of space even after the extension has been made.

2.2 Roof Opening and Closing

A roof sliding mechanism has been developed as one of the construction features of Super Wing. This mechanism is useful not only at the time of construction, but also for possible opening or closing the roof during use of the installed facility. In exhibition halls,

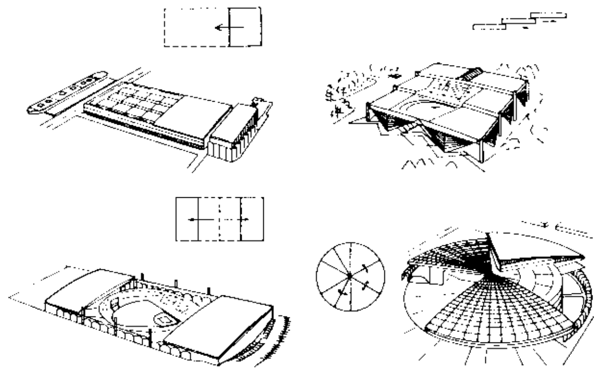

Fig. 4 Open and shut pattern

sports stadiums, and cultural facilities, lighting, ventilation and an unobstructed view to the sky can be obtained by opening or closing the roof according to weather conditions. In plants, warehouses, etc., an opening roof is effective in replacing equipment, as well as generally in the delivery of materials, shipment of products, etc.

Since PSST is basically a flat, long-span construction method, the roof opening/closing pattern is basically a lateral pull type, as shown in **Fig. 4**. Various other types, such as rotating types for exhibition halls, are also feasible.

2.3 Design Criteria for PSST

PSST is a combined steel truss structure with strands for prestressing. Its design concept therefore is not greatly different from the conventional method. Some of the significant points to be taken into consideration are described below:

(1) Influence of Prestress upon Members:
While the lower chord members are subject to the same compressive stress as the amount of prestress, no additional stress is imposed on either the upper chord or lattice members.

(2) Prestress to Be Introduced:
The amount of prestress is determined as described in paragraph 2.1.2 (1). The prestress is subject to change due to elongation or contraction of the lower chord members of the truss with load variations after installation. The ratio of this change, however, is comparatively small in the lower chord, so that the PSST structure can be analyzed based on the condition that the prestress force is a constant external load applied to the PSST.

(3) Relaxation of Prestressing Strand:
In the prestressing strands, the amount of prestress decreases with elapsed time due to the phenomenon called relaxation. Design safety should be confirmed therefore by investigating the conditions both immediately after prestressing and after relaxation (final value). Detailed descriptions are given in the following section regarding the extent of relaxation normally experienced.

3 Structural Tests

In order to investigate the assumptions related to structural design mentioned in Sec. 2, various structural tests were carried out.

3.1 Behavior of PSST When Prestress Is Introduced

3.1.1 Full-scale model structure

In order to evaluate structural behavior, a full-scale model was constructed. A plan view and the framework of the model structure are shown in **Fig. 5**. In this model, the anchors for prestressing strands were provided at two locations, one on the right and a second on the left, and a 100-t prestress was imposed at the end of the truss with 140-t applied in the middle. Prestressing strands, 12.7 mmϕ × 7 covered with polyethylene film as a corrosion preventive, were installed along both sides of the W shape steel lower chord members.

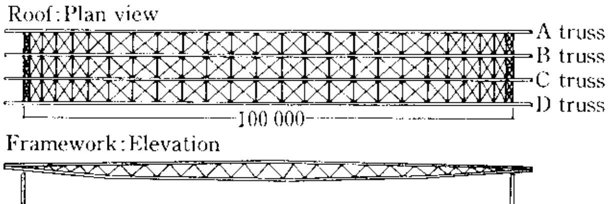

Fig. 5 Actual experimental scale mode

3.1.2 Procedure for introducing prestress

Based on calculations, the full-scale model was deformed 43 cm by vertical loading and restored 27 cm by introducing prestress. A 16-cm camber, corresponding to the difference between the deformation and restoration, was given to the truss in a workshop. To introduce prestress, the full-scale model was jacked down to the design shape as shown in **Fig. 6**. Since the prestress was introduced under these conditions, the jacking reaction force decreased theoretically with its introduction, becoming zero at the end.

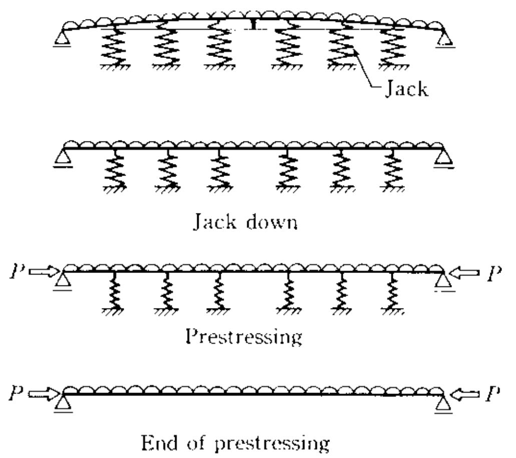

Fig. 6 Prestressing procedure

3.1.3 Stress in members when prestress is introduced

Variations in the axial load in the middle of the upper and lower chords of each truss are shown in **Fig. 7**. Initially, the upper and lower chord members were subjected to approximately a 50-t axial load by jacking down, and then as prestress increased, the axial load changed linearly, reaching the bending point at about 40 to 50-t prestress. After this bending point was reached, the axial load on the upper chord members remained almost unchanged, and only the lower chord members were subjected to axial compressive load equal to the prestress increment. The bending point mentioned above occurs where the truss is detached from the jack.

It has been proven that with PSST prestress is introduced only in the lower chord members, with no effect on the other members. Uplift on the truss, which was not supposed to occur according to the design, occurred because the actual vertical load was 80% of the design value in the A and B trusses and 70% in C and D.

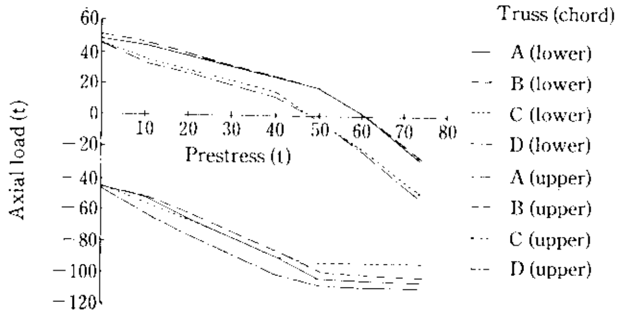

Fig. 7 Axial load in chord member during prestressing

3.2 Loading Test

Using the full-scale model, concentrated loads of up to 10 t were applied in 2-t steps to the center of truss B to investigate the behavior of PSST under additional loading. **Figure 8** shows the axial loads on the strands and on the middle of the lower chord member at the

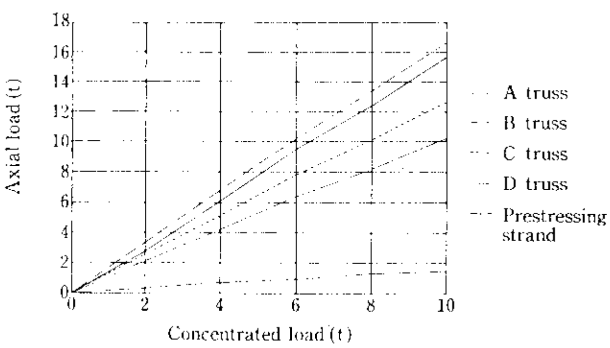

Fig. 8 Axial load in lower chord member from concentrated loading

time of loading.

3.2.1 Change in strand axial load due to additional load

The tensile load imposed on the strands due to this additional loading was only about 10% of the axial load in the lower chord member, as shown in the Fig. 8, and was not large enough to cause yielding or breakdown. It is not necessary, therefore, to consider the variations in the axial load in the prestressing strands due to variable loads such as snow and wind. Axial load due to prestressing strands may also be assumed to be a constant external load for the design of the steel truss.

3.2.2 Three-dimensional effect on truss

The additional axial load exerted on the lower chord members of truss B under the 10-t loading was calculated to be 62.5 t. The load on truss B was actually distributed to the other trusses as shown in the figure, because the structure behaves in a three-dimensional way when small girders or diagonal braces are arranged properly.

3.3 Relaxation of Prestressing Strand

The relaxation of the prestressing strands is an important problem relating to the design reliability of PSST, and thus the relaxation characteristics must be evaluated quantitatively. For this purpose, a relaxation test using prestressing strands combined with anchors (prestressing unit) and a prestressing unit in the steel truss were carried out.

3.3.1 Relaxation test of prestressing unit

(1) Test Method

The test specimen is shown in **Fig. 9**. The prestressing strands used were SWPR7A 15.2 mmϕ and 8.1 mmϕ × 7 multi-strands, both prestressing strands blue annealed and hot stretched treatment to lessen relaxation. **Table 1** shows their mechanical properties.

The methods for anchoring prestressing strands comprise the thread method, which uses a nut anchoring system, and the wedge method, which mainly uses a wedge anchoring system. Two 15.2 mmϕ strands were used for the former and 8.1 mmϕ × 7 multi-strands

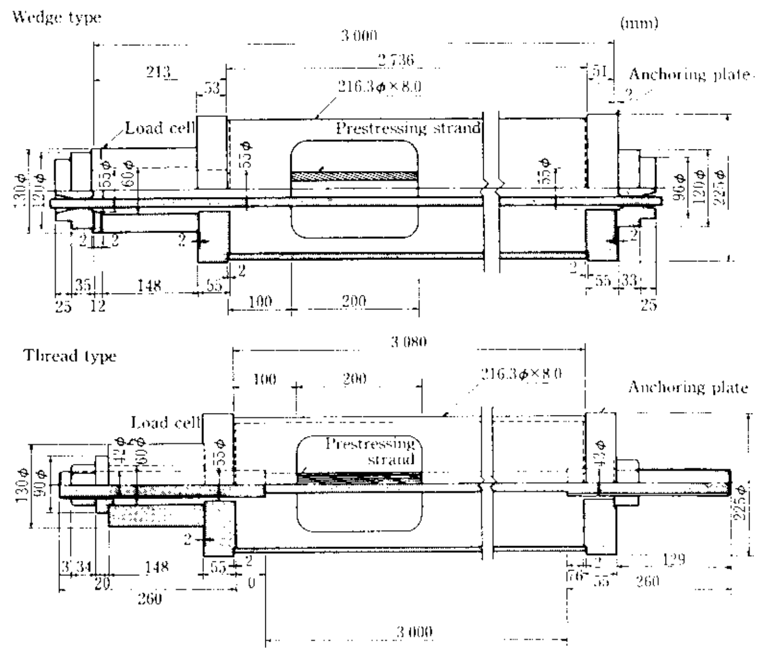

Fig. 9 Test specimen for relaxation test

Table 1 Mechanical properties of prestressing strand

Strand	Diameter (mm)	JIS specified breaking load (t)	Breaking load (t)	0.2% Proof load (t)	Elongation (%)	Relaxation (%)*1	Young's modulus (t/cm²)
15.2 mmϕ × 1	15.31	23.1	25.9	23.7	5.6	0.41	1 970
8.1 mmϕ × 7	24.60	51.0	57.4	53.0	4.8	0.43*2	1 970

*1 Value of mono-strand after 10 h

*2 Value of mono-strand (ϕ8.1)

Table 2 Test specimens for relaxation test

Method of anchor	Test specimen	P_t (t)	P_t/P_u (%)	P_t/P_u* (%)	Prestretching
Wedge	K 6	31.1	60	67	——
	K 7	36.3	70	79	——
	KP 7	36.3	70	79	0
	K 8	41.4	80	90	——
Thread	S 5	28.7	50	56	——
	S 6	34.4	60	67	——
	SP 6	34.4	60	67	0
	S 7	40.2	70	79	——
	SP 7	40.2	70	79	0
	S 8	45.9	80	90	——

P_t: Targeted prestress
P_u: Breaking load
P_u*: Specified breaking load (JIS)

were used for the latter. The length of steel pipe was determined in advance so that the test length of the prestressing strands would be 3 m in both cases. The amount of prestress introduced was 50 to 80% of the breaking load (P_u) of the prestressing strands. At 60% and 70% of this ratio, the pre-stretch anchoring (anchoring with the intended prestress after holding 5 min with a 10% increased prestress) was also applied. Specifications of the test specimen are listed in **Table 2**.

(2) Test Results

Figure 10 shows the time-related changes in the axial load of the strands; **Fig. 11** shows the amount of deformation of the anchor fittings (probably due to slip, extension, creep, etc.). **Figure 12** indicates the ratio of relaxation 1 000 h after anchoring.

In both methods, the larger the prestress, the greater the ratio of relaxation. When prestress did not exceed 70% of the breaking load, relaxation was less than 3% even after 1 000 h. For the test specimen without pre-stretch anchoring, relaxation was greater with thread method than with the wedge method. The reason for this can be seen from Fig. 11 in thread method, where deformation of the anchor fittings progressed during the early stages after introduction of prestress; the decrease in the strain on the strands corresponded to this deformation. In any actual structure, of course, the strands will be considerably longer than the 3 m in this test, and the effect of the deformation of the anchor fittings thus becomes negligible in practical use.

In a test specimen which was subjected to pre-stretch anchoring, the deformation of anchor fittings did not progress, and similarly for the case of wedge type method.

It can be assumed therefore that there is no difference between single and multiple strands if only the strands themselves are considered.

(3) Estimation of Final Amount of Relaxation

Based on these test results, the final amount of relaxation of prestressing strands was estimated. Although a number of methods have been suggested to evaluate the amount of relaxation,[1] the following formula was used to approximate the relaxation curve:

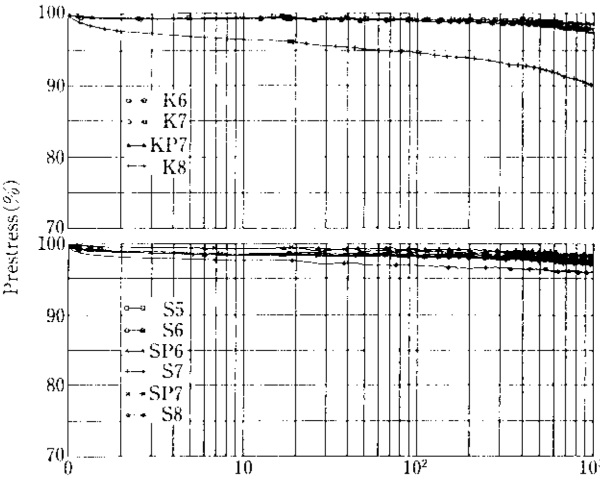

Fig. 10 Reduction in prestress

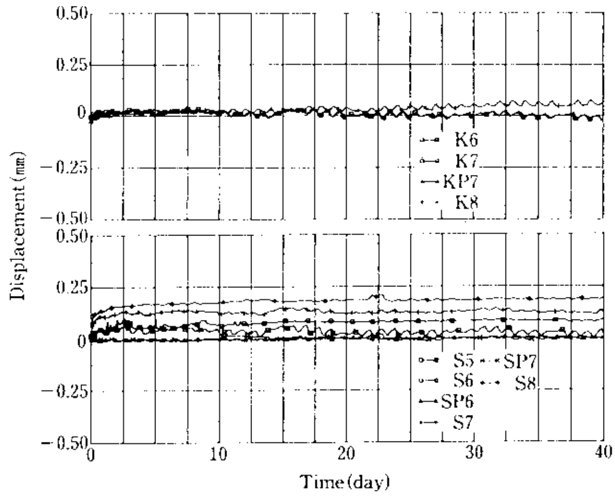

Fig. 11 Displacement of anchorage

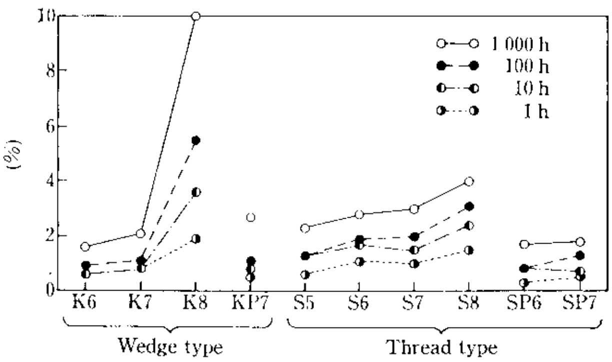

Fig. 12 Reduction in prestress

Table 3 Estimation of relaxation

Test specimen	a_0	a_1	a_2	Residual square sum	Standard deviation	Estimate (%)	
						30 years	50 years
K 6	0.669	−0.228	0.140	0.112	0.066	3.54	3.83
K 7	0.825	−0.270	0.193	0.295	0.105	5.02	5.44
KP7	0.729	−0.394	0.278	0.542	0.142	6.76	7.36
K 8	2.23	0.479	0.567	2.49	0.304	21.5	23.0
S 5	0.971	0.158	0.069	0.575	0.146	3.84	4.04
S 6	1.19	0.149	0.085	0.516	0.138	4.50	4.75
SP 6	0.444	−0.058	0.120	0.289	0.105	3.64	3.92
S 7	1.17	0.234	0.094	0.398	0.124	5.19	5.47
SP 7	0.317	0.224	0.069	0.136	0.072	3.55	3.77
S 8	1.63	0.743	0.017	0.182	0.084	6.17	6.37

$$R = a_0+a_1 \log t + a_2 \log t^2 \quad \cdots\cdots\cdots\cdots(1)$$

where R: % relaxation
t: period
a_0, a_1, a_2: constants

The results based on the assumed values are shown in **Table 3**. As shown in this table, relaxation based on both methods is less than about 5% if the prestress does not exceed 70% of the breaking load.

3.3.2 Relaxation test of prestressing unit in steel truss

(1) Test Method

The prestressing strands used were SWPR7A 8.1 mmϕ × 7 multi-strands; mechanical properties are shown in Table 1. The test specimens shown in **Fig. 13** are SW1, with a straight upper chord member, and SW2, with a curved upper chord. These two specimens were set on anchor frames and connected by members at suitable intervals in order to prevent buckling of the chord members (**Fig. 14**).

The amount of prestress introduced into the upper chord member was 35 t, which was equivalent to about 60% of the breaking load (P_u) of the prestressing strands. Prestretch was not applied in this case.

Loads of 2 t were concentrated at the ends of the truss in order to apply a stress equivalent to the dead load.

(2) Test Result

Figure 15 shows the time-related changes in the axial load in the prestressing strands. As shown in this figure, for the prestressing unit in a steel truss, relaxation loss is very slight, only about 1% even after 1 700 h. There is no difference between SW1 and SW2, and therefore the results mentioned above can be applied to the prestressing unit in the steel truss.

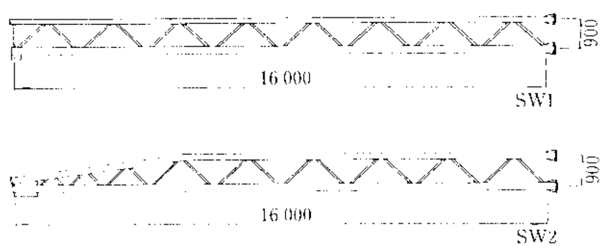

Fig. 13 Test specimens

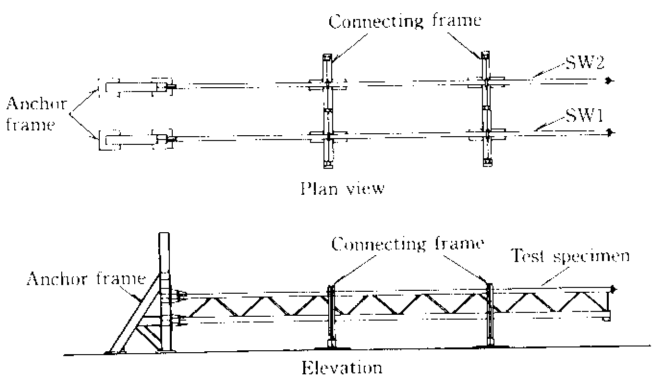

Fig. 14 Anchor of test specimens

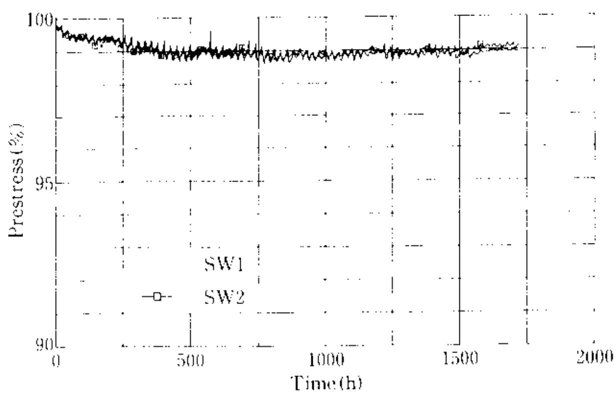

Fig. 15 Reduction in prestress

4 Investigation of Wind Resistance

Although wind resistance designs are usually based on a Japan Building Standards Act or other enforcement regulations, the coefficient of wind resistance provided in the law is established by analysis and experimental studies considering uniform laminar flow, and hence the effect of wind turbulence and building size are usually ambiguous. Wind tunnel tests, therefore, as recommended by the law, were carried out to determine the coefficient of wind pressure for the design of long span buildings, which is the objective of the Super Wing system.

Attention was paid to the following points in order to obtain the coefficient of wind pressure for the wind resistance design of Super Wing:

(1) Relationship between local conditions (coastal and urban areas) and the coefficient of wind pressure.
(2) Relationship between building size (eave height, depth and width) and the coefficient of wind pressure.
(3) Relationship between the maximum coefficient of wind pressure and wind direction.

4.1 Outline of Wind Tunnel Test

The following test method was used:

(1) Wind Tunnel Specification: A wind tunnel (2.6 m wide × 2.4 m high × 15.0 m long) owned by Shimizu Corp. was used.
(2) Air Flow in Wind Tunnel: The air flow used in the tests was a laminar flow with turbulent flow within the boundary layer. The profile index for coastal and urban areas was $\alpha = 1/6.2$ and $\alpha = 1/4.0$ respectively, and the thickness of the boundary layer was ZG × 50 cm (**Fig. 16**).
(3) Wind Pressure Model: The basic dimensions were $h = 5$ cm (height), $b = 50$ cm (width) and $d =$ 25 cm (depth). For the air current in coastal areas, 17 basic models of varying heights and depths and 2 models of varying heights with a 1/50 roof slope were used. For the air flow in urban areas, 6 models with basic dimensions and one model with a sloped roof were used. A total of 26 such models was used, as listed in **Table 4**.
(4) Measurement of Wind Pressure: The outputs from wind pressure gauges were subjected to A/D conversion with a sampling interval of $t = 0.015$ s, a sampling number of $N = 2\,048$ and an evaluation time of $T = 30.74$ s. The wind pressure was measured based on the difference in the static pressure measured in pitot tubes which were set in free flow above the model.
(5) Wind Direction: Square models were tested at 0°, 22.5° and 45° wind directions, counting counterclockwise from the 0° angle (perpendicular to the width of model). Models other than the square shape were tested with wind directions of 0° to 90° at intervals of 22.5°.

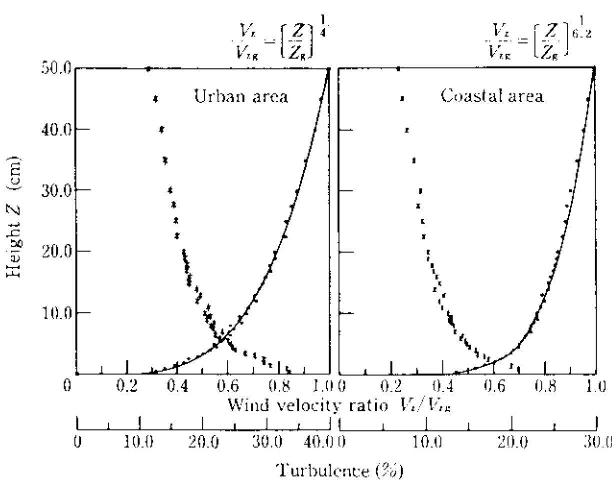

Fig. 16 Air flow in wind tunnel

Table 4 Dimensions of models (cm)

	No.	Measuring point	Width	Depth	Height	Roof slope
Coastal area	Model- 1	Roof surface	50	5	5	0
	Model- 2		50	15	5	0
	Model- 3		50	25	5	0
	Model- 4		50	35	5	0
	Model- 5		50	45	5	0
	Model- 6		50	50	5	0
	Model- 7		50	60	5	0
	Model- 8		50	75	5	0
	Model- 9	Roof surface	50	25	7.5	0
	Model-10		50	35	7.5	0
	Model-11		50	50	7.5	0
	Model-12		50	25	10	0
	Model-13		50	35	10	0
	Model-14		50	50	10	0
	Model-15	Roof surface	50	25	5-5.5	1/50
	Model-16		50	25	7.5-8	1/50
	Model-17		50	25	10-10.5	1/50
	Model-18	Wall surface	50	25	5	0
	Model-19		50	25	10	0
Urban area	Model- 1	Roof surface	50	5	5	0
	Model- 3		50	25	5	0
	Model- 6		50	50	5	0
	Model-12		50	25	10	0
	Model-15	Roof surface	50	25	5-5.5	1/50
	Model-18	Wall surface	50	25	5	0
	Model-19		50	25	10	0

4.2 Data Evaluation

The average wind pressure coefficient P_{pe} is defined by Eq. (2).

$$P_{pe} = P_M / q_r \quad \cdots\cdots\cdots\cdots\cdots\cdots (2)$$

where P_M: average wind pressure $(P - P_S)$
P: wind pressure at point of mesurement
P_S: static pressure in wind tunnel
q_r: reference velocity pressure $(1/2 \cdot \varrho \cdot U_R^2)$
ϱ: actual air density
U_R: wind velocity at eave height

The wind pressure vector which pulls the surface to the outside is defined as negative pressure ("lift"), and that acting to push the surface to the inside is considered positive pressure.

4.3 Test Results and Evaluation

Average wind pressure coefficients obtained from air flows assumed for coastal and urban areas are shown in **Fig. 17, 18, 19, 20,** and **21**, which indicate the following. (The point of measurement was located at the center of the model parallel to the wind direction.)

(1) The absolute value of the wind pressure coefficient is greater in coastal areas than in urban areas. (Fig. 17)
(2) Wind pressure distributions varied with the leeward condition of air currents burbled at the windward end.
(3) The lower the eave height of the building, the larger the absolute value of wind pressure coefficient at the windward side. (Fig. 18)
(4) The depth distribution of the wind pressure coefficients becomes greater as d/h (depth of building/height of building) increases, reaching a maximum at $d/h = 4$, and then decreasing, indicating an almost constant value when d/h is greater than 6. (Fig. 19)
(5) The properties of burbled flows as affected by both-end shapes of models becomes nearly two-dimensional at $w/h = 4$, then at $w/h > 5$, resulting in an almost constant value of the maximum wind pressure coefficient at the windward end. (Fig. 20)
(6) The wind direction resulting in a maximum wind pressure coefficient was 45° for the model 50 cm wide × 5 cm deep × 5 cm height, and 22.5° or 67.5° for the other models. (Fig. 21)

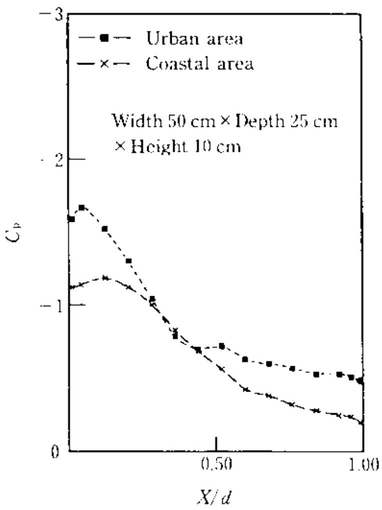

Fig. 17 Wind pressure coefficient in coastal and urban area (X: Distance from separation point)

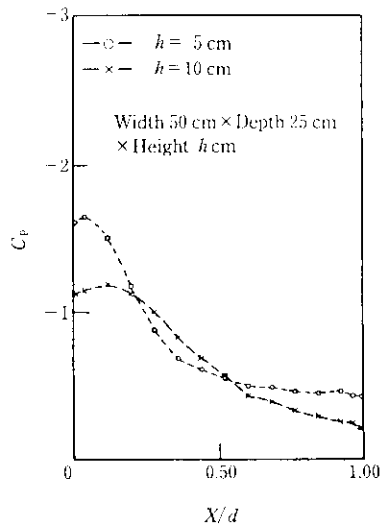

Fig. 18 Wind pressure coefficient for different building heights in coastal area (X: Distance from separation point)

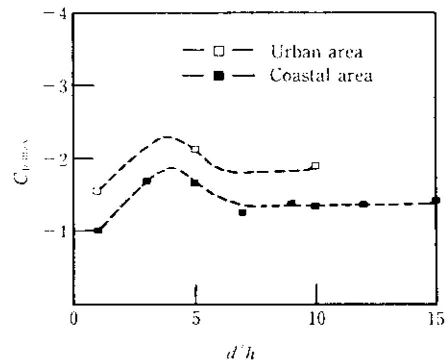

Fig. 19 Maximum wind pressure coefficient for buildings of different depths

The absolute value of the wind pressure coefficient is at its maximum at the windward end and decreases rapidly toward the leeward, probably due to reattachment of air flows burbled at the windward end. Burbled flows are affected not only by building shape, but also by the intensity of air flow turbulence which is the

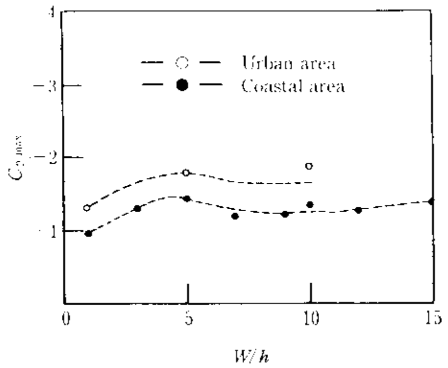

Fig. 20 Maximum wind pressure coefficient for buildings of different widths

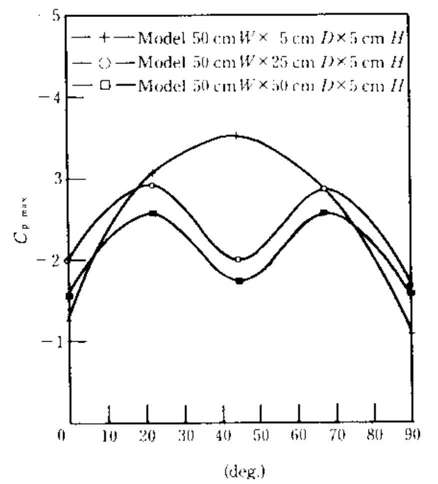

Fig. 21 Wind pressure coefficient at different wind direction in coastal area

characteristic of natural air currents. Increased turbulence develops the shear layer and promotes re-attachment of the burbled flow as well as an approach of the burble toward the windward side, thus producing a strong negative pressure on that side. It can be considered, therefore, that the wind pressure coefficient is greater in urban areas than coastal areas and also greater in buildings with low eaves, where turbulence relative to the eave height is great. It can also be deduced that the depth distribution of the wind pressure coefficient relates to the fact that the reattachment of the disturbed flow varies with building depth.

The wind pressure coefficients obtained in wind tunnel tests are experimental values of external pressure coefficients; for design use, it is necessary to consider the cumulative effect of the coefficient of internal pressure and external air pressure factors. The data from the current tests, however, is considered invaluable for an efficient, economic design of Super Wing.

5 Slide Method for Construction of Super Wing

While it is conventional practice to erect temporary scaffolds under the entire roof surface, a slide method was developed for construction of Super Wing, with the following advantages:

(1) Shorter Erection Period: With the slide method, which leaves the enclosed floor space unobstructed, it is possible to overlap scheduling of construction work, such as truss assembly, erection and finishing with machinery foundation work and equipment work, thereby shortening the start-to-finish erection period by more than 20%.

(2) Reduced Temporary Work: Trusses are assembled on the ground and a platform (as described below), and slid in succession to form the whole roof. By carrying out finish work simultaneously with assembly, temporary scaffolds are required only for the assembly space, which dramatically reduces the costs associated with temporary work.

(3) Safety: High elevation work is not necessary for the entire roof surface, but is limited to the platform required for assembly, thus improving construction safety.

5.1 Working Procedure

An outline of the work procedure for long-span structures using the slide method is shown below, based on the results obtained from a site test with a full-size model (100-m span × 15-m width) **(Fig. 22)**.

(1) Flat Assembly of Trusses at Near Ground Level: Truss members are assembled flat on a 600-mm-high stand installed on the ground surface. (step 1)

(2) Box Assembly of Trusses: Flat trusses assembled on the ground and their connecting members are correctly assembled in a box stand whose top is open. (step 2)

(3) Erecting of Trusses: Trusses assembled in box-shape are erected accurately and connected together as integrated box-shape trusses on a platform installed at eave height. In order to determine positioning accurately, the level is adjusted with jacks installed

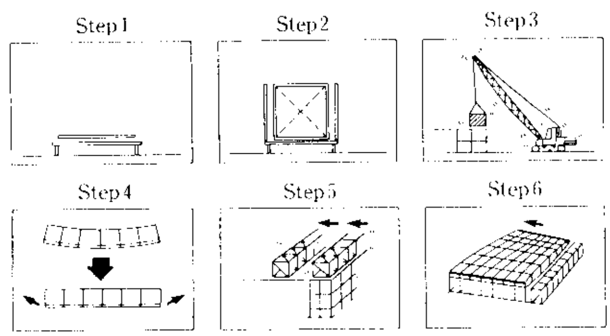

Fig. 22 Slide method

on this platform. (step 3)

(4) Prestressing Strands Tension Work: After assembling the truss into an integral body, in order to determine the finished level of roof accurately, the truss is jacked down to a proper position, taking into account the calculated amount of uplift due to the tension of the prestressing strands. Prestressing strands are then placed and prestress is introduced with jacks. The tension of the prestressing strands is controlled by a oil-hydraulic sensor and load cell method. (step 4)

(5) Slide Work: The prestressed truss is slid with jacks as far as is necessary to allow assembly of the next adjacent truss and for erecting the connecting members of the box-shaped truss. After the strands of the next truss are prestressed and the connecting members of the box truss are erected, the unit is slid to a specified position. (step 5)

(6) By repeating these five steps, steel frames for the entire roof are assembled. At the same time, roofing work, for example, installation of formed colored galvanized steel sheets, is carried out. For the site test, a roll forming machine was installed at the eave height of the roof, and simple rolls were set on tight frames to permit the bent plates to slide smoothly in order to form a long colored galvanized steel sheets from a coil. In this way, a 100-m-long stretch of roof can be covered with a single piece of roofing material.

5.2 Slide Mechanism

The roof trusses are slid on rails by pulling a wire or a rod attached to the truss ends by means of an oil-hydraulic jack. To control slide deviation of both truss ends, a device is used to correct the movement of both truss ends when the deviation exceeds a certain limit. This work is all controlled automatically by computer. In site tests when the limit of deviation was set to 10 mm, the truss was moved to the specified position smoothly.

For smooth sliding, slide friction was reduced by applying Teflon to the truss legs and also by installing stainless steel plates with a super-smooth finished surface on the rails. While the coefficient of friction depends on the surface condition of the stainless steel and the use of a surface activator, its average value during these tests was found to be between 0.05 and 0.06.

For the Super Wing construction, it is important to select the method, such as the sliding method, the uplift method or a combination of two such methods, best suited to the actual site conditions. Kawasaki Steel has an established technology for the uplift method, and now that the sliding method has also been proved to be an established technique by these current tests, it has become possible to meet more varied site conditions.

6 Conclusions

The unique structural design and construction method called Super Wing was developed, which facilitates unobstructed open space using long-span trusses. Its physical properties and construction method were studied through loading and vibration tests using a full-scale model, actual assembly tests and measurement of relaxation, and wind tunnel tests. The results obtained follows:

(1) In designing this structure, relaxation of prestressing strands should be considered. The prestressing strands relaxation will be 5% when the prestress at its introductory period is under 70% of tensile strength of prestressing strands.

(2) Construction method such as introduction of prestress and roof forming methods were established. Roof forming methods include slide method and liftup method.

(3) Wind pressure distribution of long-span structure was clarified through wind tunnel tests. In designing, coefficients of internal pressure should be considered in combination with that of external pressure.

PSST is characterized by the combination of prestressing of steel frames and the sliding construction method and offers a wide range of potential applications. This unique method has already been applied to four actual cases.

In closing, the authors wish to express their deep gratitude to Professor Igarashi of Osaka University and Professor Katsura of the Disaster Prevention Laboratory at Kyoto University, both of whom played a leading part in the development of this method, and also to Shimizu Corp. for its cooperation as partner in the joint development work.

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