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Wind-resistant Design of Cables for the Tatara Bridge

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Summary

The stay-cables of the Tatara Bridge have a much lower natural frequency than those of other bridges because of their longer length. Wind tunnel tests were conducted to study their rain vibration characteristics in low frequency ranges as well as to examine the efficiency of vibration control methods using an aerodynamic measure. As a result, it was found that indenting the surfaces of cables improves the aerodynamic stability and has less drag force than that of cables with roughened surfaces. Finally, the aerodynamic measure was applied to the cables of the Tatara Bridge.

1. Introduction

The Tatara Bridge, with a center span of 890 meters, is the longest cable-stayed bridge in the world(Fig.-1) at the completion. It connects two islands, Ikuchijima and Ohmishima, on the Onomichi-Imabari route of the Honshu-Shikoku Bridge. This bridge employs a composite structure of steel and PC: the dead-load imbalance between the center span and the side spans, caused by the side spans being shorter than the center span, is compensated by PC girders installed at the end of the side spans. The cables are anchored in two planes of a fan-shaped arrangement. Each cable consists of galvanized steel wires and is coated with polyethylene (Table-1).

The typical vibrations of cable-stayed bridges caused by wind include rain vibration, wake galloping, and vortex-induced vibration. What bridges have shared in common regarding rain vibration was that they were located in relatively flat areas and that the surfaces of cables were smooth, for example, covered with polyethylene [1), 2)]. This suggested that the Tatara Bridge may also likely be subject to rain vibration. Besides, there were reports that rain vibrations could have large amplitudes, requiring additional damping, it appeared necessary to consider damping techniques adequately in advance. On the other hand, their multi-cable arrangement tends to increase the importance of reduction of the wind load of the cables. If this large wind load(the drag force) is capable of being decreased, it will be a great help in optimizing the whole configuration of the structure. In view of these circumstances, we studied the characteristics of rain vibration and discussed not only measures for vibration control but also for smaller drag force.

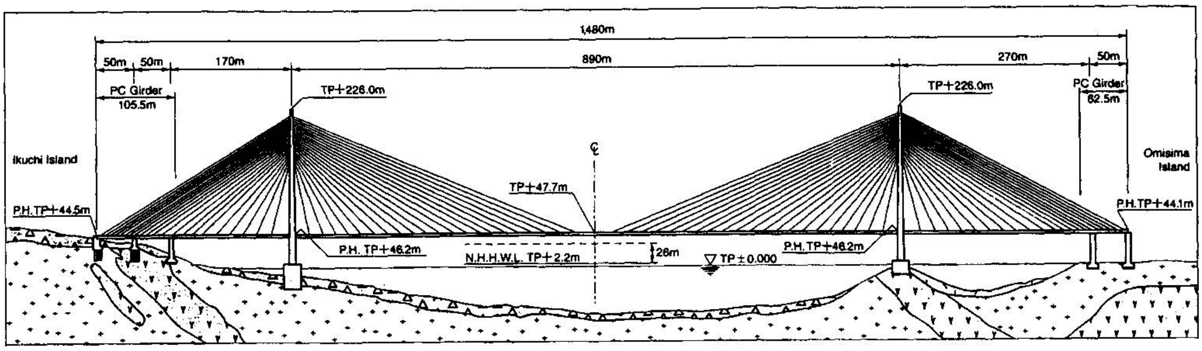

Fig.-1 General View of Tatara Bridge

Table-1 Dimension of Cables

Type	Multi-fan type (anchored in 2 planes)
Number of Cables	168 cables (21 levels)
Cable Construction	151~379 galvanized wires(ϕ7mm)/cable
Corrosion Protection	Polyethylene coating
Cable Diameter	110~170 mm
Cable Length	108~462 m
Cable Weight	5~56 ton/cable (480~1, 194 N/m)
Frequency	0. 26~1. 05 Hz (when completed, first mode)

2. Investigations into Characteristics of Vibrations

2.1 Summary of Wind Tunnel Tests

The Tatara Bridge Cables are characterized by an extremely low natural frequency compared with those of conventional cable-stayed bridges, because of very long cable length resulting from the long span. The longest cable of this bridge was expected to have a natural frequency of about 0.26Hz while the lowest natural frequency was about 0.5 Hz in the case of other cable-stayed bridges. Thus, wind tunnel tests [3] were conducted for the primary purpose of clarifying the vibration characteristics in low frequency ranges, which was an unexplored field.

The cable model used for the wind tunnel test was a full size rigid body consisting of a 12-m long 155-mm diameter, covered by polyethylene in which a steel pipe was inserted, and both ends of which were supported by springs for the purpose of studying vibration characteristics. A full scale model was used, because it ensured similarity requirements such as adhesion characteristics of water to the cable surfaces and those of water rivulets. Rain was simulated by a spray nozzle at the exit cone of the wind tunnel and an auxiliary nozzle at the top end of the cable.

2.2 Vibration Characteristics of Tatara Bridge Cables

The wind tunnel test revealed the following characteristics (Fig.-2).
1)Rain vibration is generated even in a low frequency range (0.26 to 0.54 Hz).
2)Rain vibration occurs in the wind velocity range of about 6m/s and 12m/s, and it is generated at the lowest wind velocity when the relative angle is 45°.
3)Of the two velocity ranges, the vibration in the higher wind velocity range occurs with rivulets formed on both the upper and lower surfaces of the cable while the vibration in the

lower wind velocity is generated with rivulets formed only on the lower surface of the cable.
4)If the structural damping of the cable is 0.02 or so in terms of logarithmic decrement (δ), rain vibration is suppressed by local vibration.
5)Turbulent flow has some damping effect with respect to rain vibration.

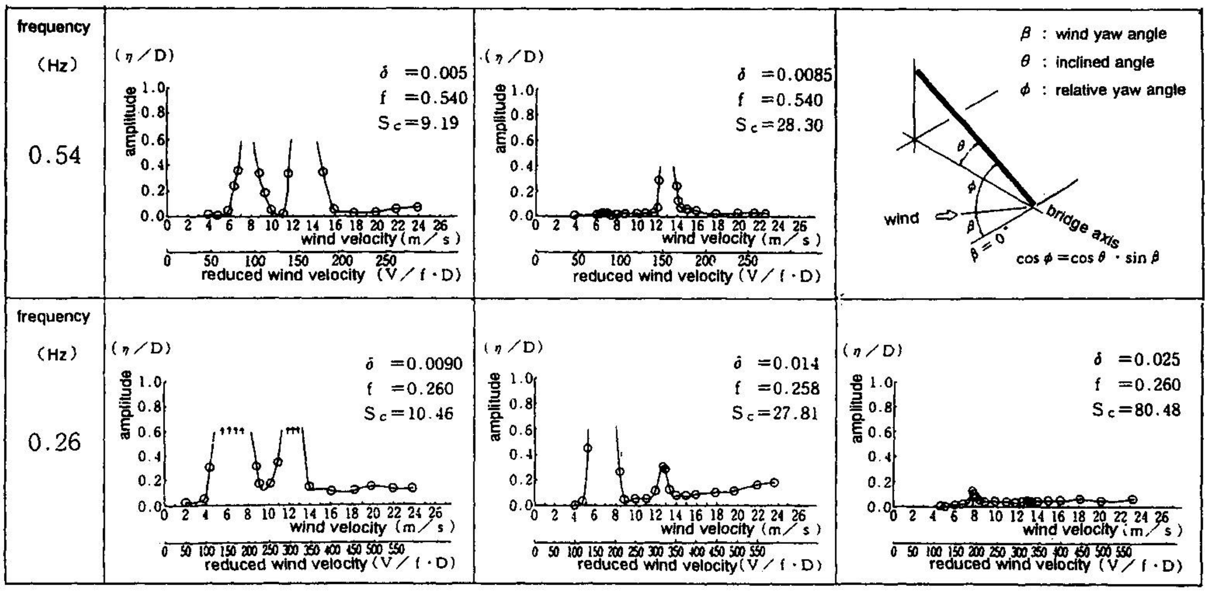

Fig.-2 Vibration Characteristics of Cable

3. Discussion of Vibration Control Measures

3.1 Application Problems of Vibration Control Measures to Tatara Bridge

As measures to control rain vibration, three methods are actually in use: tying the cables with wires, increasing structural damping of the cables by installing damping devices, and improving aerodynamic characteristics of cables by giving deformation on their surfaces. With the Tatara Bridge, an aerodynamic countermeasure was applied to the cables for the following reasons.

The tying-cable method is not effective on vibrations because wire connections form other mode shape nodes of the vibrations. Moreover, the method intended for all the high-order modes is not practical, because the countermeasure should be taken to the 11th mode for the longest cable. Regarding the installing damping device method, damping effects can be estimated by analytical calculation to a certain degree. In the case of the Tatara Bridge, the structure of the equipment and maintenance of mountings present problems, because the long cables make the size of the dampers larger and their mounting locations higher. Furthermore, this method is not desirable from an aesthetic point of view.

The aerodynamic method, which processes the surfaces of cladding material to directly suppress the exciting forces acting on the cables, does not require secondary equipment. Therefore, it is free of the problems posed by the structures of equipment, which are often encountered in the case of the tying-cable method and installing damping devices method. Various aerodynamic countermeasures have been proposed and already applied for a few bridges. In the East-Kobe Bridge [4], parallel protuberances were applied to the stay cables covered with polyethylene. Helical strakes were added on the surface of the stay cables in the Normandy Bridge. U shaped groovings were cut on the surface of the stay cables coated by polyethylene in the Yuge Bridge.

However, the Tatara Bridge had the following problems. Aerodynamic countermeasures were apt to cause a greater drag force at the design wind velocity than that of a smooth surface. For example, the drag coefficient of the cable with parallel protuberances is 1.2 at the design wind velocity, while 0.7 for a cable with a smooth surface. In the design of the Tatara Bridge, a smaller drag force is preferable, because wind load for cables with smooth surfaces comes up to about 30% of total wind load. If a cable having a larger drag force were applied to the Tatara Bridge, that would affect the structural design of the deck or the tower section. Therefore, it was quite necessary to investigate any aerodynamic countermeasure to develop a cable section with a smaller drag force and, at the same time, better vibration suppressing effects in a low frequency range.

3.2 Discussion of Aerodynamic Countermeasure

To investigate the effect of the surface configuration of cable on aerodynamic properties, three-component balance tests [5)] were carried out. The drag coefficient of a body with a circular section is a function of the Reynolds number, $Re=VD/\nu$, where V is the flow velocity; D is the representative length; ν is the kinematic viscosity. Fig.-3 shows the results of the experiment that compared drag characteristics between a cable surface having uniform roughness and that having discrete concave pattern (indented cable). In both cases, the roughness was 1% of the cable diameter. The indented cable (Photo.-1) showed a different trend from the cable with uniform roughness. Its drag coefficient (C_D) was 0.61 at a critical Reynolds number of 1×10^5. Within the range of measurements up to a Reynolds number (Re) of 5.5×10^5, equivalent to a wind velocity of about 55 m/s, the drag coefficient remained approximately constant, less than 0.7. This proved that if a cable surface is roughened discretely, drag characteristics almost equivalent to those of a smooth cable (with a circular cross section) can be obtained in the range of design wind velocity.

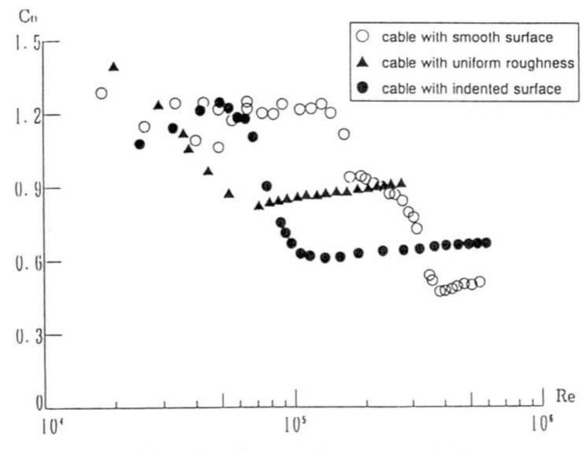

Fig.-3 Drag Characteristics

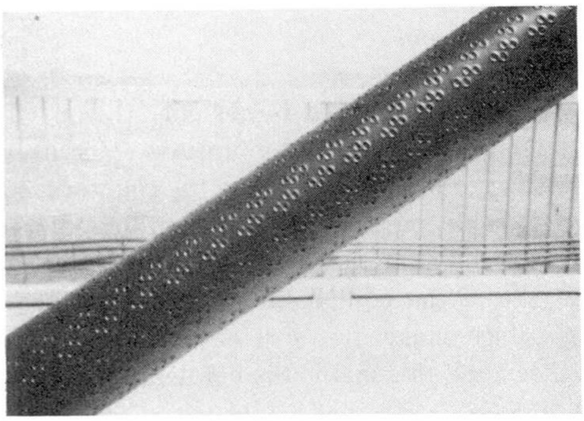

Photo.-1 Indented Cable

The characteristics of the cable having an indented pattern was analyzed through measurement of pressure distribution(Fig.-4). As for the cable with a smooth surface, in the subcritical range, the location of the separation point, θ, was about 80° for the Reynolds number (Re) of about 0.9×10^5, and the pressure coefficient on the rear surface was almost constant, showing that thorough flow separation takes place at the rear surface. In the supercritical range at a Re number of about 5.5×10^{5}, the separation point moved backward an angle θ of about 100°, and the static pressure on the rear surface was restored because of turbulence mixing, narrower wake width and so forth. On the other hand, with a cable having the indented pattern, almost no difference was noted in the pressure coefficient at Re number of 0.9×10^{5} and 5.3×10^{5}. The separation point (θ) was located at an angle of about 110°. This proves that the indented cable already reached a supercritical state at a wind velocity of about 10 m/s, which agreed well with the results of the drag coefficient measurement. With the indented cable, a negative pressure peak was observed at an angle θ of about 80° in the supercritical state. This negative pressure is estimated to suppress the formation of rivulets on the upper surface: such rivulets are one of the causes of rain vibration. This indicates that the indented cable is able to suppress rain vibration by increasing apparent Reynolds numbers.

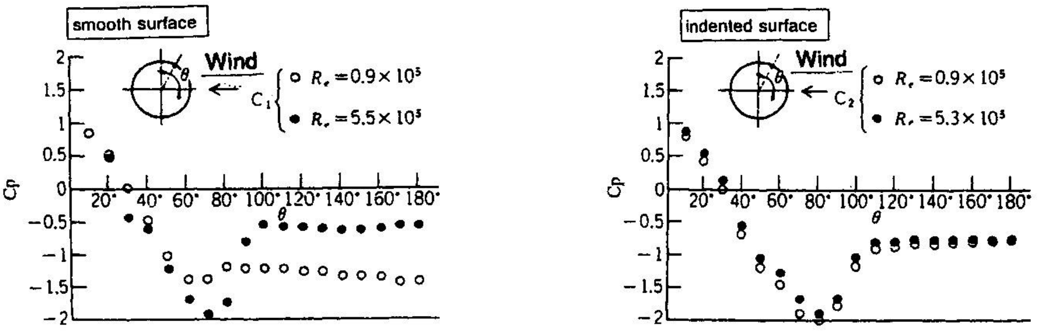

Fig.-4 Pressure Distribution of Cables

3.3 Results of Wind Tunnel Tests

As a measure to control rain vibration aerodynamically without increasing the drag coefficient, a cable surface having discrete concave roughness (indented cable) was considered. The drag coefficient of the indented cable was about 0.7, which was almost equal to the design drag coefficient of a cable with a smooth surface. In order to learn the rain vibration suppressing effects of the indented cable, vibration tests under simulated rainfall were carried out. The method used for the wind tunnel test was the same size as that indicated in 2.1. The results of tests are shown in Fig.-5. The aerodynamic characteristics of the cable are summarized as follows.

1) There was no evident vibration in a high wind velocity range in which vibration was observed with smooth cables. Some vibration was observed in a low wind velocity range, at a wind velocity of about 6 m/s.
2) Vibration in a low wind velocity range, which occurred under a limited wind velocity range, was able to be suppressed when the in-plane (vertical) structural damping (δ) of the cable was increased to 0.02 or so.
3) The vibration tended to be suppressed in turbulence flow, and the indented cable had more suppressing effects than those of smooth surfaces.
4) Frequency dependency was observed in the generation of vibration: rain vibration became less prone to occur in high frequency ranges and no evident vibration was observed in the frequency range above 1 Hz.

It is considered that the vibrations at a high wind velocity range were caused by the rivulets on upper surfaces of the cables and that suppression effects was observed as the location and the width of the rivulets changed. On the other hand, as the vibrations at a low wind velocity range were caused by the rivulets forming on the lower surfaces. It appears that the surface treatment alone was not sufficient in controlling vibration. Through rainfall experiments, it was confirmed that the indented cable had rain vibration suppressing effects in a high wind velocity range. It is considered that the indented pattern had effects on raising the apparent Reynolds number and making the flow separation point move backward. The vibration observed in a low wind velocity range around 6 m/s could be reduced negligibly to a low level in the actual bridge, considering the turbulent flow at the site. Besides, with indented cables, vibrations are limited to low-order modes below 1 Hz if they ever appear, making it easier to take installing damping measures than with cables having smooth surfaces.

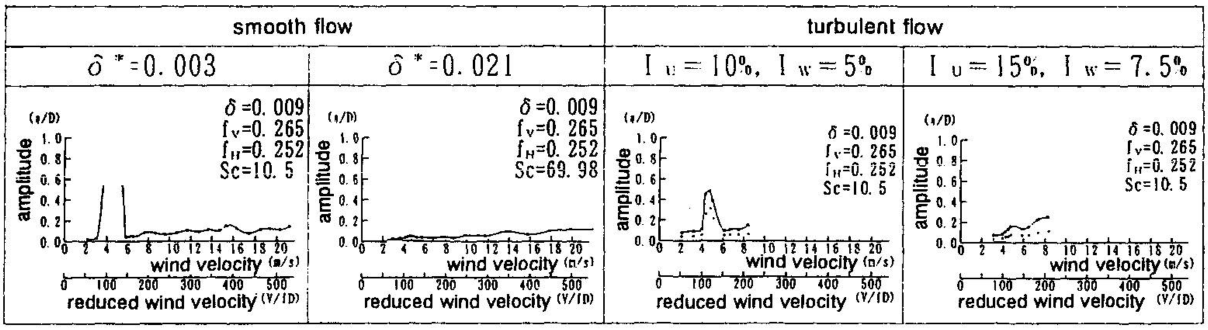

Fig.-5 Test Results of Indented Cable

4. Conclusions

In this study, vibration characteristics of the Tatara Bridge Cables were examined and wind-resistant designs using an aerodynamic countermeasure were discussed. The results obtained by the study are summarized as follows.

1) The indented cable was adopted for the Tatara Bridge as an aerodynamic measure to control rain vibration. It appears that indented cables have a sufficient suppressing effect on rain vibration, considering the turbulent flow at the site.

2) The indented cable has a drag coefficient almost equal to that of a cable with a smooth surface, resulting in decreased wind loading.

3) A structure has been employed that will allow installation of dampers, to prepare for the worst.

4) Finally, the behavior of the cables on the actual bridge has been watched since the construction phase. And no rain vibration has been noted up to now.

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Vibration Control of the Main Towers of the Akashi Kaikyo Bridge

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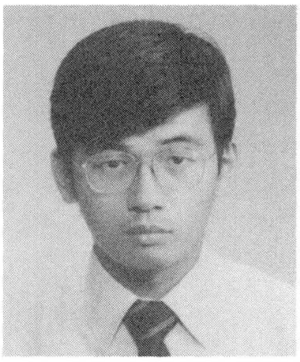

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Summary

The main towers of the Akashi Kaikyo Bridge are 300m tall, and have very flexible feature. The natural frequency of the towers becomes relatively low, and they are easy to vibrate under the wind even after completion of the bridge.
To control the vibration of the towers due to the wind, shape of cross section was improved and Tuned Mass Dampers(TMDs) were installed.
In this paper, the outline of the vibration control of the towers of the Akashi Kaikyo Bridge is reported. Also, the result of vibration tests and field observation is reported.

1. Introduction

The main towers of the Akashi Kaikyo Bridge are 300m tall and 100m taller than the towers of existent suspension bridges. So these towers have very flexible feature and the vibration of the towers due to the wind, not only during construction but after completion of the bridge, is one of the most important issue for the safety of this bridge.
Honshu-Shikoku Bridge Authority (H.S.B.A.) has conducted various investigations, including wind tunnel tests, for many years, and chosen improved cross section and additional damping devices as the control method against wind induced vibration of the main towers of this bridge.
During construction of the tower, measuring instruments were installed on the tower, and vibration tests and field observation were conducted.
In this paper, the design procedure of the vibration control of the towers of the Akashi Kaikyo Bridge is reported. Also, the result of vibration tests and field observation is reported.

2. Outline of the Vibration Control Method

2.1 Outline

To control the wind induced vibration, the cruciform shape of cross section was chosen for the tower shaft and some kinds of mechanical damping devices were installed on the tower.

2.2 Shape of Cross Section

Based on the results of various wind tunnel tests, the cruciform shape of the cross section was selected for the tower shaft. Size of the cut-off at corner varies according to the height of the tower.

With this cross section, the amplitude of out-of-plane vibration becomes lower, and the divergent torsional vibration at high wind velocity is suppressed.

Fig. 1 shows the general plan of the towers.

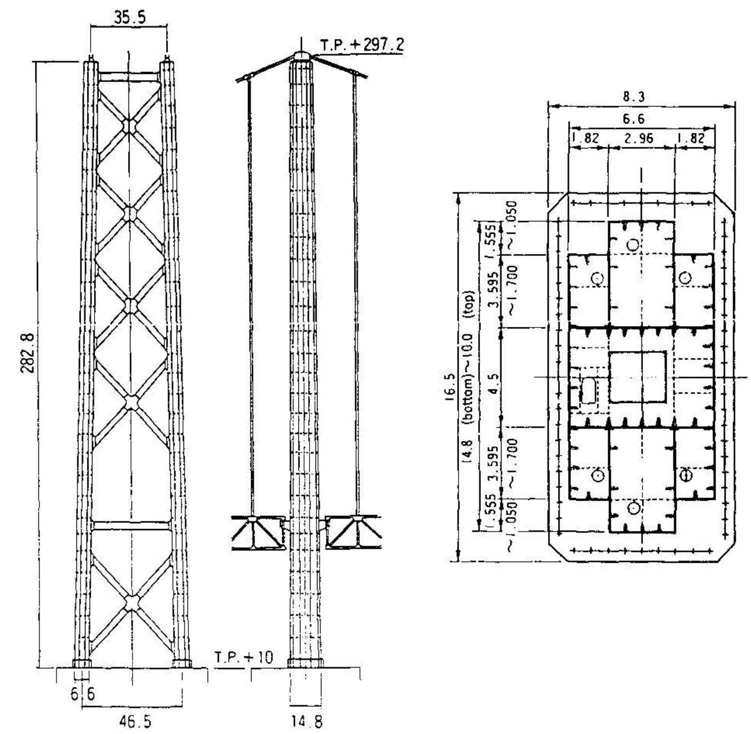

Fig. 1 General Plan of the Towers

2.3 Damping Devices

According to the wind tunnel test with 1/86th large scale model, the amplitude of the vortex induced oscillation by the wind slower than the design wind speed exceeds the allowable amplitude of vibration, and some kinds of mechanical damping devices are needed to reduce the amplitude of vibration.

As the damping device, Tuned Mass Dampers (TMDs), Tuned Liquid Dampers (TLDs), and Friction Dampers, etc. were examined, and Tuned Mass Dampers are chosen from the reliability and cost.

3. Damping Device

3.1 Outline of Design of Damping Device

Wind tunnel tests with 1/86th large scale three dimensional model were conducted for the towers both during construction and after completion of the bridge, and the response of the tower against the wind was obtained.

Based on the results of wind tunnel tests, the resonance wind speed of the vibration and the relationship between the damping of the tower and the amplitude of the vibration were examined. Considering the stress in the member of the tower and the safety of construction work, the condition of vibration control was decided.

3.2 Design Procedure of Damping Device

As the damping device, Tuned Mass Dampers(TMDs) are selected, considering the reliability and cost. During the early stage of construction of the cables, Semi-Active Dampers are installed at the top of the tower.

To design the TMDs, the tower and TMDs are assumed as Two-Degree-of-Freedom (TDOF) System and this system can be treated as the system subject to harmonic loading, because the vibration which should be controlled is vortex-induced oscillation.

Fig.2 shows the procedure to design the TMDs. At first, mass, frequency, and damping of the TMD were assumed. Then, the analysis of TDOF system was conducted and total damping of this TDOF system was obtained. Frequency and damping of TMD were adjusted until total damping of TDOF system satisfied the design damping. This procedure was repeated until minimum weight of TMD was obtained.

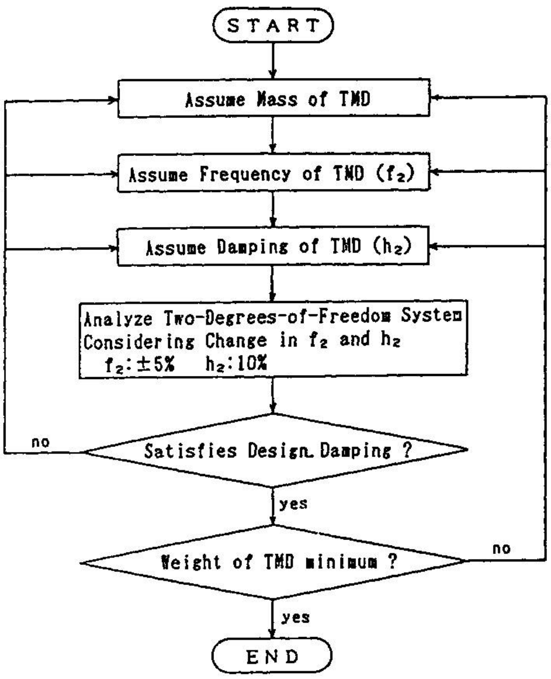

Fig. 2 Design Procedure for TMD

Table.1 shows the TMDs during construction and after completion of the bridge. Fig.3 shows the location of TMDs. The damping devices for each step of construction is as following.

3.3 Damping Devices during Construction of the Towers

During construction of the towers, TMD-E1,E2 against vibration of long period and TMD-E3,E4 against that of short period are installed at the top of the erection crane. Also, TMD-1,2 inside the tower, which are used as the dampers after completion of the bridge, are used as the additional damping.

3.4 Damping Devices during Construction of the Cables and the Girder

During construction of the cables and the girder, TMD-1,2 inside the tower are adjusted to control the vibration. And additional TMDs (TMD-3) are installed at the top of the tower. Also, Semi-Active Dampers are installed at the top of the tower to control the vibration during early stage of construction of the cables.

3.5 Damping Devices after Completion of the Bridge

As the damping device after completion of the bridge, TMD-1 for out-of-plane vibration and TMD-2 for torsional vibration are installed inside the tower.

During Construction of Bridge		
Frequency (Hz)	Damping Devices	Weight of TMDs
0.6 ~ 1.3	TMD-2 (inside Tower)	114 ton
0.3 ~ 0.6	TMD-1 (inside Tower) TMD-3 (at the top of Tower)	84 ton 21 ton
~ 0.3	Semi-Active Damper (at the top of Tower)	

→

After Completion of Bridge		
Vibration Mode	Damping Devices	Weight of TMDs
Torsional (around 0.74 Hz)	TMD-2 (inside Tower)	114 ton
Out-of-Plane (around 0.44 Hz)	TMD-1 (inside Tower)	84 ton

Table.1 TMDs during Construction and after Completion of the Bridge

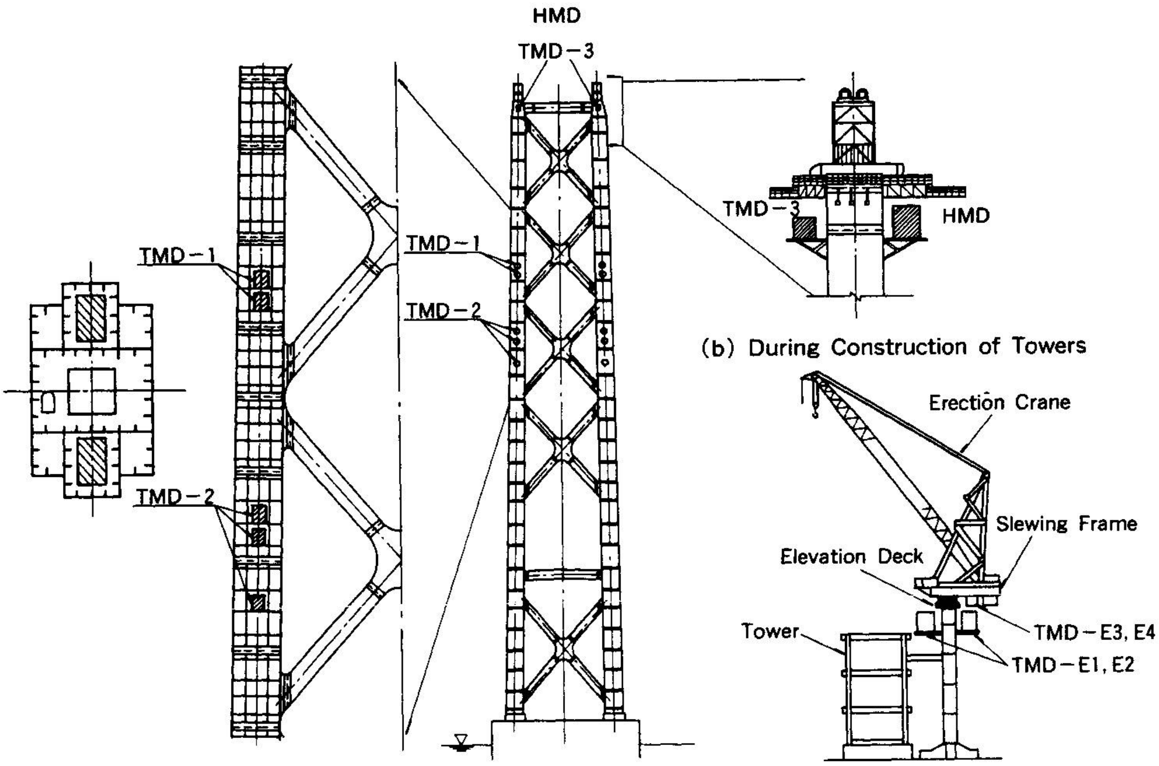

Fig.3 Location of Vibration Control Devices

4. Vibration Test and Field Observation

4.1 Vibration Test

Vibration test was conducted after completion of the tower. The characteristic of the free standing tower without TMDs and the effect of TMDs were examined. The Semi-Active Dampers at the top of the tower were used as oscillators.

Table.2 shows the frequency and mode shape of both measured and calculated values, and they have good agreement.

Vibration Mode	Natural Frequency (Hz)		Mode Shape (—— Calculated, • Measured)
	Calculated	Measured	
Out-of-Plane Vibration 1st Mode	0.127	0.126	
Out-of-Plane Vibration 2nd Mode	0.677	0.673	
Torsional Vibration	0.473	0.471	

Table.2 Natural Frequency and Mode Shape of Free Standing Tower

Table.3 shows the damping (logarithmic decrement) of the tower for each mode of vibration. The damping of the tower for 1st out-of-plane vibration is a little smaller than that specified in design standard, but damping for other mode are bigger.

The additional damping with TMDs satisfies the design requirement for each mode.

Vibration Mode	Test No.	AMD	TMD1	TMD2	TMD3	Damping δ		maximum amplitude (cm)
						Measured	Design	
Out-of-Plane Vibration 1st Mode	1	×	×	×	×	0.0067	0.0100	62
	2	P	×	×	×	0.028	0.0244	28
	3	A	×	×	×	0.105	0.0752	19
	4	P	O	O	O	0.038	0.0244	21
	5	A	O	O	O	0.111	0.0752	21
Out-of-Plane Vibration 2nd Mode	6	×	×	×	×	0.038	0.0100	1.3
	7	×	×	O	×	0.080	0.0454	0.9
	8	P	O	O	O	0.096	0.0454	1.0
Torsional Vibration	9	×	×	×	×	0.028	0.0100	5.3
	10	×	O	×	O	0.075	0.0418	3.0
	11	P	O	O	O	0.075	0.0418	3.0

Table.4 Damping (Logarithmic Decrement) of Free Standing Tower

4.2 Field Observation

During construction of the tower, measuring equipment was installed on the tower and the field observation of the behavior of the tower was conducted. The purpose of the field observation is to observe the vibration of the tower and the movement of the TMDs to confirm the design assumption and the effect of vibration control.

Since the measuring instruments were installed on the tower, a lot of records have been collected about the behavior of the tower. For the example of these records, a record about vortex induced oscillation is reported here.

Fig.4 shows the relationship between wind velocity and amplitude of vibration due to the wind of around perpendicular direction. Also the result of the wind tunnel tests are shown in Fig.4.

Considering the record of field observation, it was confirmed that the vibration characteristic of the towers agrees the assumption for the wind tunnel test and effect of the TMDs satisfies the design requirements.

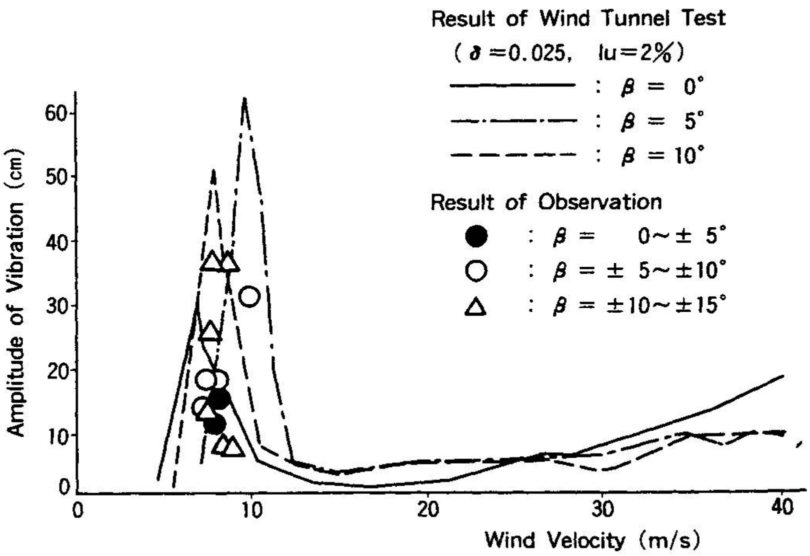

Fig.4 Relationship between Wind Velocity and Amplitude of Vibration

5. Conclusion

In this paper, the design procedure of the vibration control and the result of field observation and vibration tests about the towers of the Akashi Kaikyo Bridge is reported.

Based on the results of various wind tunnel tests, the cruciform shape of the cross section was selected for the tower shaft and Tuned Mass Dampers are installed to reduce the amplitude of vibration..

Vibration tests and field observation were conducted during the construction of the towers, and it was confirmed that the vibration characteristic of the towers agrees the design assumption and effect of the TMDs satisfies the design requirements.

The authors hope that the design conducted for the vibration control of the towers of the Akashi Kaikyo Bridge and result of the vibration test and field observation will contribute the wind resistant design for the same kind of structure in the future.

6. Acknowledgements

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Design and Construction of the Akashi Kaikyo Bridge's Superstructure

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Summary

This paper describes some technical features of the Akashi Kaikyo Bridge's superstructure as follows: 1) For the towers, wind resistant design was required. The towers were fabricated and erected with high vertical accuracy. 2) For the cables, erection was done by prefabricated strand (PS) method using newly developed high-tensile strength wires. The dehumidification system was newly developed to increase the life of cables. 3) For the stiffening girder, wind resistant design was done using a newly built large boundary layer wind tunnel facility. 4) Because of some displacement of foundations due to the Southern Hyogo Earthquake, design and fabrication of the stiffening girders had to be modified in order to adjust to the new configuration of the Bridge.

1. Some problems to be overcome for the superstructures

Followings were essential problems to be solved for the superstructures of the Bridge.
1) The Akashi Kaikyo Bridge is a suspension bridge spanning an international navigation channel that is more than 1.5 km wide and has a sea traffic of 1,400 ships a day. Then, the erection of superstructures had to be done not to interrupt the sea traffic.
2) Because of its long-span and high-rise structures, the bridge is very flexible and is susceptible to wind. Then, the wind induced dynamic oscillation of the tower and the stiffening girder had to be suppressed thorough the wind resistant design.
3) Some displacement of foundations occurred by the crustal movement due to the Southern Hyogo Earthquake in January 1995, when the cable erection was almost over. To the change of the Bridge's configuration, some modifications of the design and fabrication of the stiffening girders had to be done.

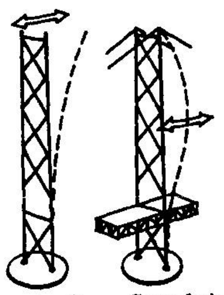
2. Technical features of tower

2.1 Wind resistant design of the tower

The tower is a steel flexible tower, with the height of 287 m. From experiences so far, the tower vibrations by the wind vortex due to the wind from the transverse direction had been a problem to be overcome, during erection. Since the primary natural frequency of the tower of the Bridge is half as low as that of other 1 km class Honshu Shikoku Bridges, it was feared that vibration would occur below the design wind velocity of 66.7 m/s, not only during erection but also even after completion of the bridge.

Table. 1 Comparison of aerodynamic properties of towers

Name of Bridge	Tower height H (m)	1st bending frequency (Hz)	Resonant wind speed (m/s)
Free standing			
Akashi Kaikyo	287.6	0.131	10.0
Kita Bisan Seto	169.5	0.256	16.5
Completion			
Akashi Kaikyo	287.6	0.465	41.8
Kita Bisan Seto	169.5	1.126	86.0



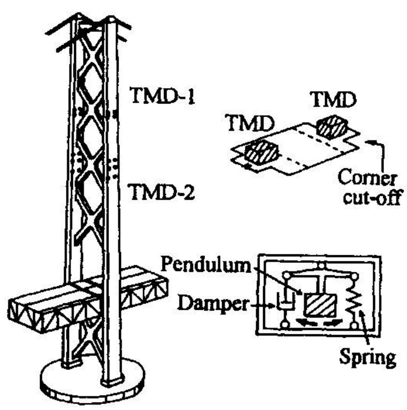


Fig. 1 Installation of Tuned Mass Damper

The vibration can be controlled either by aerodynamic improvement of the cross section or by installing damping devices. For aerodynamic improvement of the cross section, fabrication of corner cut-offs in the cross section was judged to be most effective to suppress vortex-induced oscillation, through several wind tunnel tests.

At the same time, damping devices had to be installed to suppress the amplitude of vortex-induced oscillation to the allowable level.

The chosen damping devices were a group of tuned mass dampers (TMD), for economical and structural reasons. TMD consists of a hanging pendulum mass, a spring and oil damper, as shown in Fig. 1. TMD-1 (84 t) is a damping device to suppress the vibration of the 1st bending mode, and TMD-2 (114 t) is for the 1st torsional mode for the completed bridge. Both devices were installed inside of the tower shafts.

2. 2 Fabrication and erection

To fabricate the tower, the tower was divided into 30 blocks vertically, and each block was further divided into 3 cells, whose weight did not exceed 160 tons, the capacity of the crane. As the vertical accuracy, the tolerance at the top of the tower was specified to be less than 1/5000 × (tower height). To realize this accuracy, in the shop, each block had to be fabricated with the accuracy of less than 1/10,000. Then, in the shop, 3 cells were temporarily assembled, and the sectional planes were ground and polished to the required flatness, using a

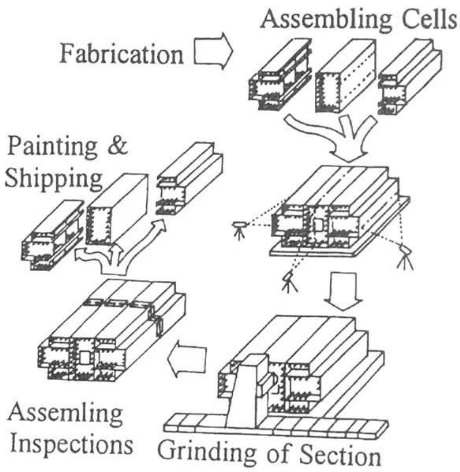


Fig. 2 Fabrication of tower shafts

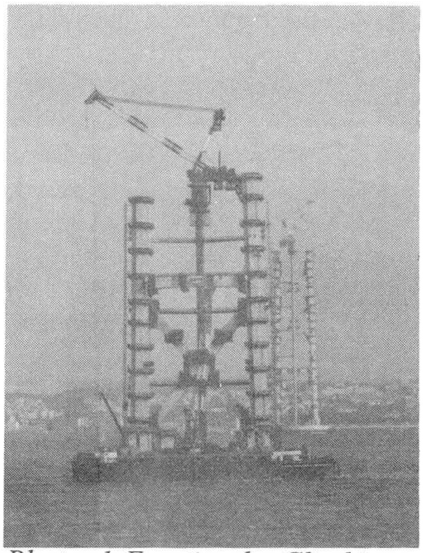


Photo. 1 Erection by Climbing Tower Crane

specially made large-size cutting & grinding machine. Afterward, the blocks were again separated into 3 cells and transported to the site. At the site, a climbing tower crane was adopted to minimize the erection period. After it had laid each block, the crane jacked itself up to the next level and hoisted up another one. The actual error at the top of towers was 39 mm at 2P and 29 mm at 3P.

3. Technical features of cable

3. 1 Development of high tensile strength wire

For more than 50 years, galvanized steel wire, with a diameter of 5 mm and with tensile strength of 155-160 kg/mm^2, had been used for the cable of suspension bridges, due to the established production technology and stable quality. The size of the cable is determined by the sag ratio and the allowable stress of wires. Using 160 kg/mm^2 class wire for the Bridge, the diameter became too large, exceeding the experience of cable erections so far. So the, double cable system (2 cable/one side) could not be avoided. But the double cable has problems, such as increased dead load, complicated structure of the girder and long erection period. To solve those problems, development of high strength galvanized steel wire was required. To increase the strength of galvanized steel wire, the following three means were considered.

1) Increasing the degree of processing during wire drawing
2) Adding small amounts of other elements
3) Controlling strength loss due to the heat reaction during galvanizing

Table. 2 Comparison of chemical ingredients of galvanized wire

	160 kg/mm^2 class	180 kg/mm^2 class
C (%)	0.75 ~ 0.80	0.80 ~ 0.85
Si (%)	0.12 ~ 0.32	0.80 ~ 1.00
Mn (%)	0.60 ~ 0.90	0.60 ~ 0.90

As a result of the investigations, low-alloy steel wire with the addition of Si proved to be the most effective way to raise the tensile strength to 180 kg/mm^2. The properties of the developed 180 kg/mm^2 class wire are shown in Table. 2. The newly developed wire showed almost the same or better quality than the conventional wires.

3. 2 Erection of the cable

To minimize the erection period, the prefabricated parallel wire method (PS method) was adopted. The composition of the cable was shown in Fig. 3.

The cable erection was started by carrying a pilot rope from shore to shore. The conventional way to carry a pilot rope is to pull a rope with floats by a tug-boat or to pull a rope by a tall crane ship. But these methods required sea traffic to be halted, so it was decided to pull the rope by the helicopter, so as not to interrupt the sea traffic on the international navigation channel. For the helicopter, the pilot rope had to be light, strong and easy to handle. So, a poliaramid fiber rope, with a diameter of 10 mm, weight of 0.0917 kg/m and tensile strength of 4700 kg, was used.

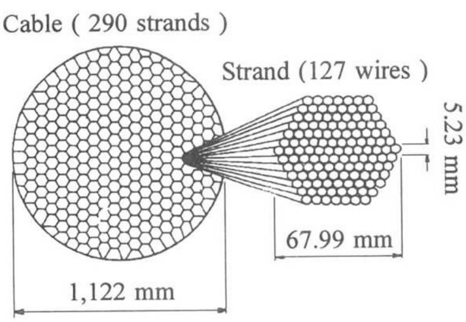


Fig. 3 Cross section of cable

The pilot ropes were connected with steel ropes, and were pulled for the replacement to the stronger ropes. These works were repeated, and the system to haul cable strands were completed. Using this system, catwalk ropes were erected with floors on them. Cable strands were erected as follows: 1) strand reels were transported to the yard of 1A, 2) each strand was pulled by a strand carrier on a hauling system along the catwalk. Cable sag was measured and adjusted during the night when the temperature is stable.

Photo.2 Carrying a pilot rope by helicopter

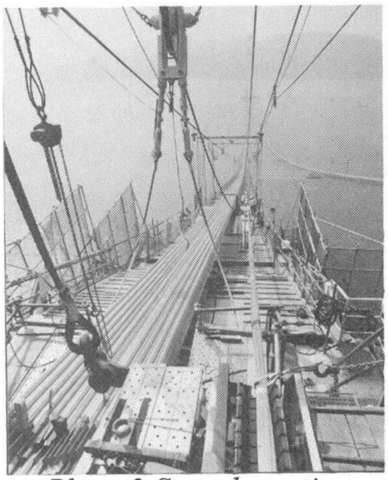


Photo.3 Strand erection by hauling system

To minimize the erection period two hauling systems were used for one cable.

3. 3 Corrosion protection method of cable

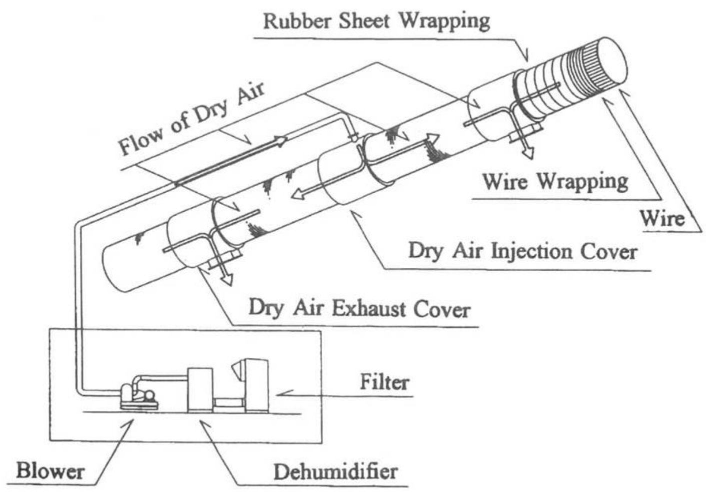


Fig. 4 Dehumidification system of cable

The conventional corrosion protection system for cables consists of a surface paste on the wire, wrapping wire and top coating. But the waterproofing provided by this system is not always satisfactory due to surface paste aging and thermal cracks developing in the top coating. Through several studies, it was found that the atmosphere environment inside of the cable can be improved by injecting dried air into the cable. In this system, a cable surface is completely wrapped by both wires and rubber sheets. Dry air is injected from a cable cover and is exhausted from another cable-cover about 140 meters apart from the injection cover. The pressure within the cable bundle is planned to keep within 0.03 atmospheric pressure so that the rupture of the seals and sheathing materials does not occur.

4. Technical features of stiffening girder

4.1 Wind resistant design of the stiffening girder

Due to its flexible structure, vibration, especially flutter due to wind, was the most important problem at the design stage of the stiffening girder. In the design code, it is specified that flutter must not occur under the wind speed of 78 m/s in the wind tunnel test within the attack angle from -3 deg. to +3 deg..

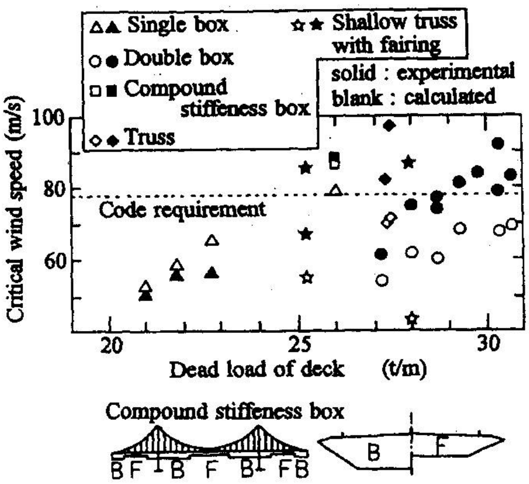


Fig. 5 Girder types and their flutter speed

To determine the type of the stiffening girder, several types of girders as shown in Fig. 5 were investigated. Fig. 5 also shows the relationships between the onset wind speed of flutter and deck weight for the investigated girders. From these results, the truss girder and compounds stiffness box girder were selected as prospective types. The compound stiffness box girder is a bridge system that arranges high-tortionally-stiffened girders around the tower and aerodynamically-well flat girders at the central portion of the bridge. From the comparison of these two types of girders, truss was finally selected due to its ease of its erection on the international navigation channel, because the erection of the truss can be done by not using the navigation channel.

The dynamic behavior of suspension bridges due to the wind has been investigated mainly by wind tunnel tests using sectional models so far. For the Akashi Kaikyo Bridge, however, a full model test in a large wind tunnel facility was required for the following reasons. 1) The deflection of the bridge due to wind is large. 2) Along the bridge axis with the length of 4 km, there is a large variation of wind properties. 3) The effect of the main cables cannot be neglected. 4) The effect of the turbulent flow cannot be neglected. Therefore, we built a large boundary layer wind tunnel facility with width of 41 m, height of 4 m, and length of 30 m. As a result of the experiment of a scale of 1/100, it was confirmed that the required wind resistancy could be obtained by installing some gratings on the road deck and a vertical stabilizing device along the truss girder as shown in Fig. 6.

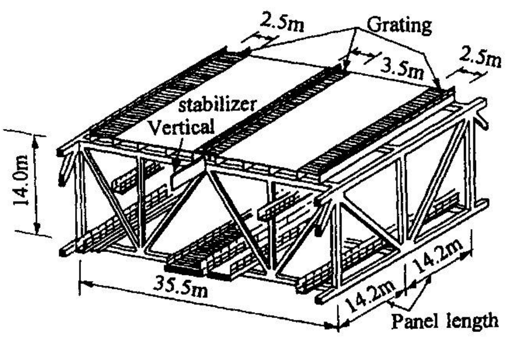


Fig. 6 Stiffening girder of the bridge

4.2 Erection of stiffening girder

Girder erection was started by lifting large-block truss girders using a 3,500 - 4,100 ton floating crane at six places, front sides of anchorages and sides of the two towers. As, these areas are located outside of the navigation channel, those methods could be applied. At those six places, pre-assembled plane truss members with a length of 28 meters were lifted up on

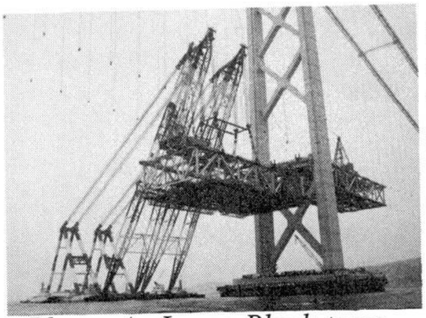


Photo. 4 Large-Block truss girder erection

the deck, carried to the front of the erected girders, and connected to them. By adopting this erection procedure, it was possible not to interrupt the sea traffic. The direction of girder's cantilever-out was as follows: 1) Center span: from two towers to the center of the straits. 2) Side spans: from each anchorage to the tower. This erection procedure was selected so that the inclination of the erected girder did not exceed 6 %, within which stability of several machines on the girder could be secured. It took 13 months from the start to the closure of the stiffening girder.

4.3 Effect of the Southern Hyogo Earthquake

On January 17, 1995, just after the cables had been erected, the Magnitude 7.2 Southern Hyogo Earthquake occurred. Its epicenter was only 3.5 km east from the center of the Bridge.

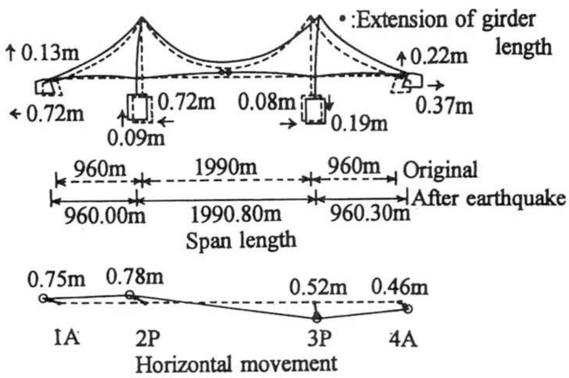


Fig. 7 Displacement of the Bridge after the earthquake

After the earthquake, displacement of foundations by crustal movements was found. As a result, the basic configuration of the Bridge changed like Fig. 7. Then modified design of stiffening girder was absolutely required soon after the earthquake. There was almost no damage on the completed structures. At that stage, fabrication of some girders had just started, but fabrication of suspenders had almost finished. As for the configuration of the Bridge, the horizontal change of angle along the bridge axis was found to be small, less than 0.04 degrees at a tower. Those angles were judged to be absorbed by the expansion joints.

The elongation of spans were 0.8 m at the center span and 0.3 m at the Awaji side outer span, and cable sags became shallow. Changes of span length were not negligible, so the length of girders had to be changed at unfabricated girders. Then, we decided to increase the length of the last two girder sections of the center span by 80 cm, and also the length of the last girder section of the Awaji side span by 34 cm respectively. The location of cable bands and suspenders were also changed. The additional stress of the structure due to the change of the bridge configuration was calculated through three-dimensional structural analysis, and was found to be small.

5. Concluding Remark

For the design and construction of the Akashi Kaikyo Bridge's superstructure, various investigations were carried out. Because of its long-span and high-rise structures, and its site location over the busy sea traffic, a lot of studies had to be done to select the adequate construction methods, and to improve the structural design methods. Through these studies, we could develop new technologies like high-tensile strength wires, wind resistant design method, new corrosion protection system for cables, and reasonable construction methods. At the same time, we could overcome the problem of the change of bridge's configuration after the earthquake. We hope that our experiences of the Bridge will be utilized for longer span bridges in the future.

Construction of Akashi Kaikyo Bridge Foundation

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Summary

The Akashi Kaikyo Bridge is supported by two anchorages both positioned on each side of the 4 km width strait and by the two tower foundations located in the deep-water region. Owing to the heavy load of the superstructure and being that the bearing stratum is deep, the volume of concrete cast in each foundation exceeds 200,000 m^3, totally amounting to more than 1.4 million m^3 as a whole. The harsh physical conditions of the Strait such as swift tidal currents exceeding 8 knots and its heavy maritime traffic in which more than 1,400 ships navigate a day, made certainty, safety as well as readiness as the subjects for consideration in executing work.
This paper gives a description of the work in deep-water slurry wall, high fluidity concrete, installation of caissons, scour protection, underwater concrete which were developed to solve the problems stated above.

1. Foreword

The Akashi Strait is 4 km in width, 110 m in maximum depth on the bridging route in which the tidal velocity reaches up to 8 knots, and a navigation lane of 1,500 m width is set up in the center. The Akashi Kaikyo Bridge as shown in Fig. 1 is a suspension bridge with a center span of 1,991 m and an overall length of 3,911 m. Foundation work conducted were for 2 anchorages at both ends of the strait and 2 tower foundations on both sides of the navigation lane. From among these four foundations, the 1A anchorage and 2P tower foundation, in which a large amount of work was involved shall be described.
The 1A anchorage located in shallow water was, after coffering and filled, soil guarded by an underwater slurry wall of 85 m in diameter, 76 m in depth, and then excavated to the bearing ground 64 m below the surface. After pouring 260,000 m^3 of roller compacted concrete in the interior, 230,000 m^3 of high fluidity concrete was cast for the anchorage structure.
The 2P tower foundation located in deep-water zone is a cylindrical shape foundation 80 m in diameter, 70 m high. After excavating the seabed from 45 m to 60 m below the sealevel with a grab bucket dredger, a steel caisson fabricated on a dry dock was installed. Then, scour protection composed of filter units and riprap was executed in a range of 240 m in diameter around the caisson, and desegregating underwater concrete of 270,000 m^3 was deposited in the caisson interior.
The work had progressed favorably since commencing in March 1988, the 1A work was completed in September 1992, and 2P work in June 1992.

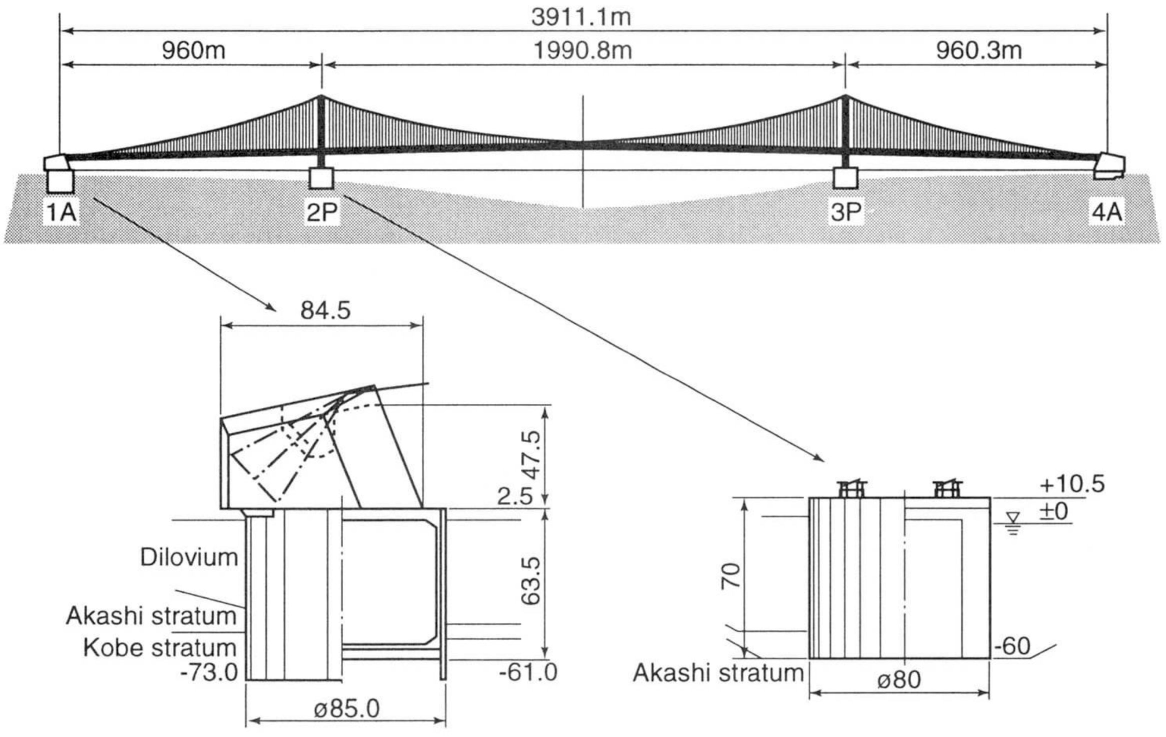


Fig. 1 Profile of Akashi Kaikyo Bridge

2. Work Execution of 1A Anchorage

2.1 Deep-water Underground Slurry Wall

The foundation of 1A anchorage was constructed by excavating the reclaimed ground 64 m below the sealevel. The underground slurry wall, which is 2.2 m in width, 85 m in diameter and a depth of 76 m, serves as the soil stopper during excavation work. Since the structure during construction has to resist with circumferential compressive force against the enormous earth and water pressure, it was necessary that the structure should be made as circular as possible, and water stoppage capability improved.

Photo 1 Work of Underground Slurry Wall

The excavator, as shown in Photo 1, a horizontal multi-bit-drill with a wall thickness of 1.5 to 3.2 m and digging length of 3.2 m, was used. At first, 23 preceding blocks, each consisting of 3.2 – 2.1 – 3.2 m elements (total length of 8.5 m) were constructed setting apart 3.2 m in distance. Latter elements of 3.2 m were fabricated between the preceding blocks to form a polygon of 92 angles. Also, when digging of latter elements, about 20 cm of the preceding blocks was cut off to allow adhesion of concrete so that a vertical accuracy of 1/1,000 is maintained. The extent of leakage of water from the joint of elements during digging of internal soil showed only oozing, demonstrating that water stoppage was satisfactory.

2.2 High Fluidity Concrete

The anchorage structure was constructed on the upper part of the foundation by casting to the interior of the precast concrete wall. The volume of concrete deposited in this structure amounted to 140,000 m^3, and in order to withstand the tensile force of 1.2 million kN applied from the main cable to the foundation, the interior was arranged with anchor frames and innumerable reinforcements.

The subjects in executing work were the suppression of heat cracking of concrete and the rapid work method to shorten the construction period. For this reason, low-heat cement with fine granulated blast-furnace slag and fly ash added was used, as well as precooling was applied by replacing part of mixing water with ice. Also, the plane was divided into 5 blocks, and 3 m width slots were provided between the blocks to accelerate natural heat radiation for relieving tensile stress.

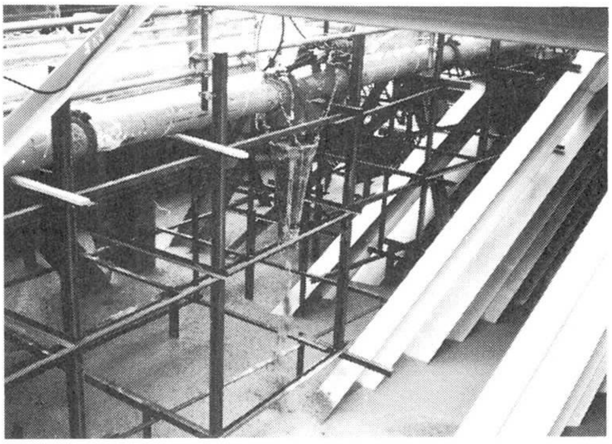


Photo 2 Casting of High Fluidity Concrete

The cement cast exceeded a maximum of 1,900 m^3 per day. Moreover, as anchor frames and innumerable reinforcements were arranged in a confined area, difficulties were in levelling and compaction of concrete. To slove these problems, high fluidity concrete was developed by replacing a portion of fine sand aggregate (1,500 kg/m^3) with pulverised stone mixed with highly effective plasticizer. This concrete possesses a self levelling character which spreads horizontally by its own weight, as well as required material segregating resistivity. Casting of concrete is shown in Photo 2. Concrete, when discharged from valves provided at an interval of 10 m on the conveying piping, will flow in by self-weight and fill the interior of the casting form. About 20 of these valves are provided in each block, and which will open or close automatically at an interval of several minutes to allow uniform discharge over the entire area.

3. Work Execution of 2P Tower Foundation

3.1 Installation of Caisson

The caisson, which becomes the form for shaping the main tower foundation, is a steel structure with an outer diameter of 80 m, 65 m in height and weights 19,000 metric tons. For the purpose of towing, it is a double-wall structure with the inside diameter of 56 m, and the sections of buoyancy on the outer side are partitioned into 16 sections to prevent the decline of stability caused by free water. As shown in Photo 3, the caisson was towed by 16 tug boats into the site during slack tide, and moored to the sinkers which were installed in advance. The mooring work, coupling the mooring cables of sinkers to those of winches on the caisson, must be finished within 2 hours. But it is no easy matter since the mooring cable is a wire rope of 120 mm in dia. Therefore, a quick joint, which couple the cables within a few seconds, was developed to facilitate

Photo 3 Mooring of Steel Caisson

mooring operation. Also, it is necessary that the winch rewinding the mooring cable is of low tension high speed winding capacity of 200 kN by 30 m/min. during mooring operation, as well as high tension low speed winding capacity of 4,000 kN by 2 m/min. at time of positioning. Thus, a special winch combined with a drum winch which performs the former work and a linear winch for the latter was developed.
The work of installing caisson was divided into tow steps. One is guiding the caisson to position by operating 8 winches, and another is lowering and setting the caisson on the seabed using 32 water sealing pumps. As both operations needed a higher accuracy positioning, georgmeters were placed at two points on land of level terrain to radio transmit the position to the caisson. The height was surveyed by installing ultrasonic bathymeters at four locations on the caisson. Aside from positioning data, more than 300 kind of information such as tidal variation, winch tension, water level of each section and the like, would be required. The various information was processed by computers, and arranged comprehensively to display on CRTs. The data was displayed in a cycle of 2 seconds allowing to grasp information by real time. The system was remarkably effective in installing the caisson, achieving the resulted positioning accuracy of less than 5 cm.

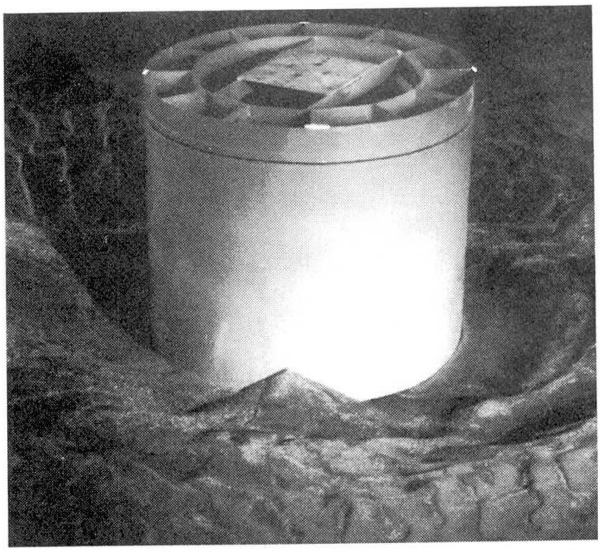


Photo 4 Scouring without Countermeasure

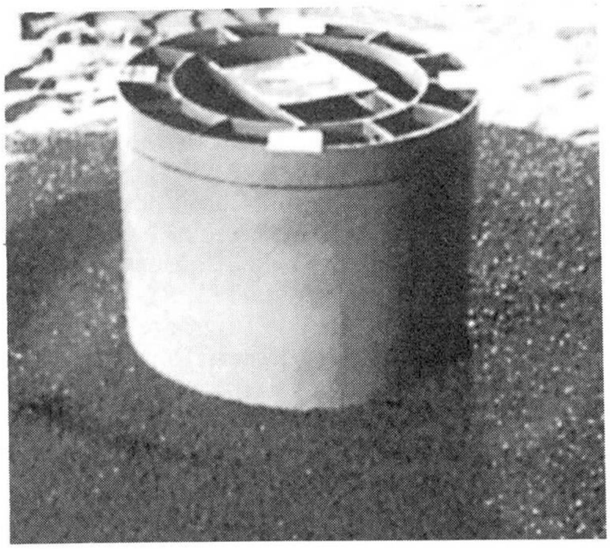


Photo 5 Scouring with Countermeasure

3.2 Scour Protection

The periphery of 2P is an area of strong tidal currents where maximum water velocity exceeds 7 knots. When the caisson was installed, accelerated currents or horse-shoe eddy generated around the caisson. Scouring began to advance from about a tidal speed of 4 knots and scour the foundation ground as shown in Photo 4. Basically the peripheral ground had been overlaid with ripraps, each weighing about 1000 kg, to prevent scouring as shown in photo 5. However, it was confirmed that, around the caisson, a sucking phenomenon of ground soil through gaps of riprap aggregate would occur if the thickness of layer was inadequate. It was found that scouring could be prevented by installing filtering layer with the thickness of 2 m, in a range of 10 m around the caisson in short term, then covering with ripraps of 8 m thick on the top. The filter unit is a netbag, weighing about 1 metric ton, filled with crushed stones the size of 30 to 150 mm dia. These filter units were put together like a grape arbor and quickly installed by a floating crane. Also, the surrounding in an area of 240 m dia. was covered with riprap to a thickness of 3 — 10 m. After this, according to a continued investigation extending over 7 years, scouring was locally found at the outmost peripheral, but is currently stabilized maintaining a satisfactory condition.

3.3 Underwater Concrete

Desegregated underwater concrete, in which desegregating admixture and superplasticizer are added to ordinary concrete, was deposited. This concrete by the workings of desegregating admixture is of high desegregating resistivity as shown in Photo 6, as well as favorable fluidity by addition of superplasticizer. As shown in Photo 7, a special plant barge was employed, equipped with 2 plant units, each with an output capacity of 90 m^3/h and a storage facility for storing 9,000 m^3 of material to cast concrete. After one batch of concrete was fully produced in about 9 days, the concrete was cast for 3 consecutive days, and this cycle was repeated 30 times in total.

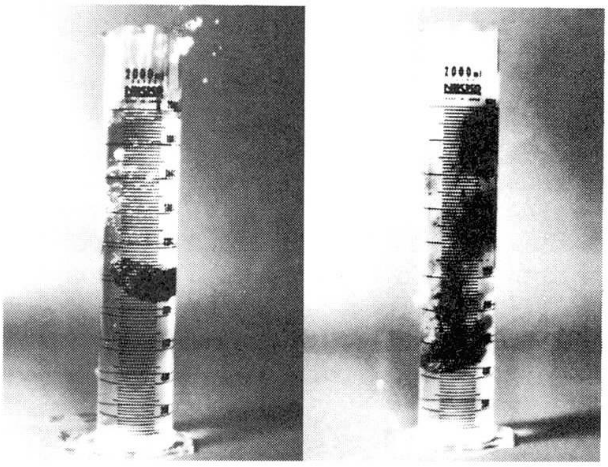


Photo 6 Comparison of Underwater Desegregation

The caisson which becomes the form is a double-wall cylindrical structure, so the area to be cast is divided into two blocks. In order to alleviate temperature stress, concrete was cast to the inner block first, then to the outer block. The total area of the inner block, which its diameter is 56 m, was concurrently cast with concrete using 24 casting pipes by a 3.5 m lift, repeating this operation 14 times. Each construction joint was treated with the newly developed underwater green-cut robot. On the other hand, the outer block is partitioned into 16 sections, each section was cast one by one, at a time from 60 m up to 5 m down below sealevel.
Due to the 270,000 m^3 mass concrete, it was necessary to mitigate temperature stress. The volume of unit cement was reduced to 320 kg/m^3 and 50 % of water was replaced by ice to maintain the temperature at cast to less than 20 °C. Also, low heat cement, which is a mixture of ordinary Portland cement, fine granulated blast-furnace slag and fly ash in the ratio of 16: 54: 30, was used. As a result, internal maximum temperature was below 50 °C, and cracking had not occurred. It was found that boring core collected from the foundation was fully adhered at the construction joint with a mean strength of 2,500 N/cm^2 with a standard deviation of 200 N/cm^2, which was of sufficient strength against the design value of 1,800 N/cm^2.

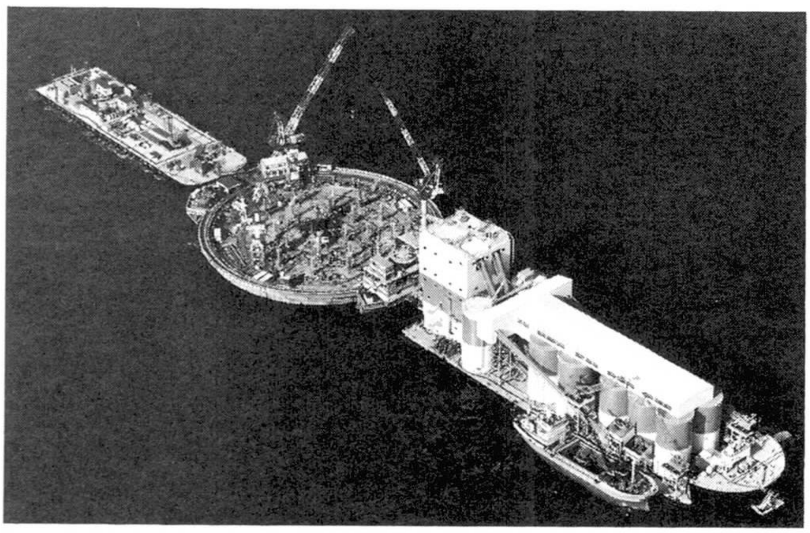


Photo 7 Condition cast Underwater Concrete

4. Conclusion

In executing work of the Akashi Kaikyo Bridge substructure, the scale of work, physical conditions, social environment, etc. were those which surpassed the standard of ocean civil engineering in present Japan, by all work had progressed smoothly. The primary factors to this success should be attributable to facilities having sufficient capacity functioning reliably, accurate and prompt information control systems, and the ability and skills to master those facilities and systems. Knowledge which were acquired from this work are enumerated below.

[1] Building of the underwater slurry wall polygonal and of quasi-circle, the soil-guard wall resisting earth pressure by the circumferential compressive force is effective in reducing wall thickness.

[2] Improvement in vertical accuracy of slurry wall, and cutting off the portion of preceding element to improve adhesion of concrete with the latter element will ensure excellent water stoppage.

[3] A portion of fine aggregate replaced with pulverized stone as well as adding high effective AE plasticizer to provide self levelling and segregating resistivity in high fluidity concrete, enable to fill the form without levelling and compaction.

[4] A low tension high speed drum winch and high tension lowspeed linear winch combined, can be controlled of the mooring cable length by 1 cm order, and is effective in improving positioning accuracy.

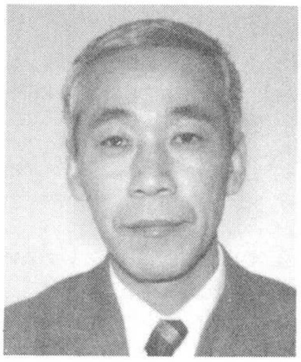
[5] Work information control system is exceedingly effective which displays by real time the level position, distance to the seabed, water level of each section, speed and direction of tidal current at time of laying the caisson, which needs to be accomplished within a short period of time during slack tide.

[6] Quick installation of filter units is feasible by a floating crane for measures against initial scouring. Scouring can be prevented by conjointly covering the surface with riprap.

[7] Controlling the volume of concrete by adequately arranged casting pipes, will allow to lift up the total area uniformly, in casting desegregating underwater concrete.

[8] By adequately eliminating marine snow and laitance sedimentation on the surface of desegregating underwater concrete will allow to integrally continue casting.

Erection of the Tatara Bridge's Superstructure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Summary

The Tatara Bridge (hereafter referred to as 'the Bridge') is located at the middle of the Onomichi-Imabari Route, the most westerly route among the three routes of the Honshu-Shikoku Bridge Project. It is a cable-stayed bridge connecting Ikuchijima Island (Hiroshima Prefecture) and Ohmishima Island (Ehime Prefecture), and the Bridge has a total length of 1,480 meters and a center span length of 890 meters. When completed, it will be the world's longest bridge in terms of scale, excelling its sister bridge Normandie Bridge (located in France with a center span length of 856 meters).
This paper reports on the design and construction of the superstructure of the Tatara Bridge.

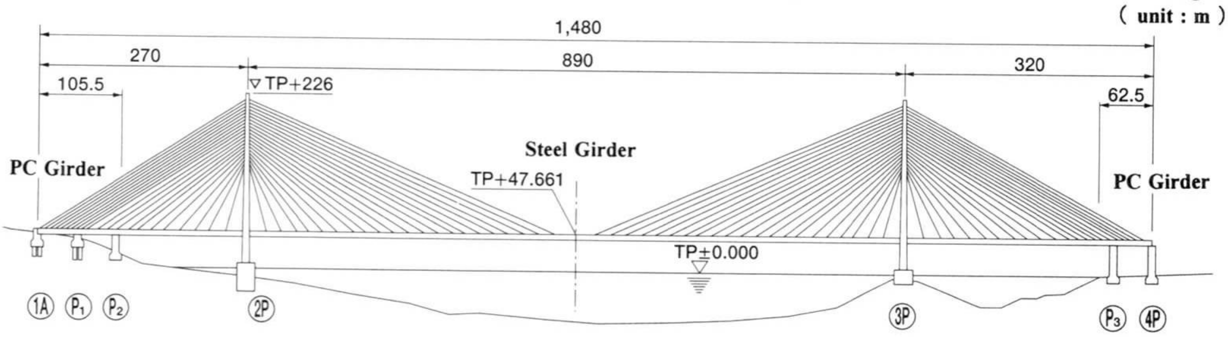

Fig. 1 General View of the Tatara Bridge

1. Design of the Bridge

In the initial design, the Bridge was planned to be a suspension bridge. Under the plan, great impact of topography was anticipated because the anchorage point on the Ikuchijima Island side was proximal to the mountain ridge at its back. However, a cable-stayed bridge design was eventually selected because it does not need anchorage, resulting in less topographical influence. Furthermore, we could assure sufficient design and installation, with subsequent technological advances, and the fact that a cable-stayed bridge was no more costly than a suspension bridge was in our favor.

Since the center span of the Bridge is too long compared with the side span, countermeasures against negative reaction occurring at the ends of the side spans were necessary as was the enhancement of overall rigidity. Prestressed concrete girders were placed at the ends of each side spans (for 1A-side: 105.5m, for 4P-side: 62.5m), and a steel girder was adopted for the entire remaining parts of the bridge to make it a composite cable-stayed bridge.

Table 1 Volume

Tower	Tower	12,110 t
	Accessory Facilities	450 t
	Anchor Frame	480 t
Girder	Steel Girder	15,860 t
	PC Girder	6,610 m3
	Accessory Facilities	450 t
Cable	Cable	3,640 t
	Socket	310 t

The cable system is double-plane with a multi-fan of 21 rows of cables. The girder is a flat box type, and its height and width are 2.7 meters and 30.6 meters respectively. Bicycle and pedestrian ways of 2.5 meters in width are extended on both sides of the 4-lane roadways of 20.0 meters in width.

2. Designing the superstructure

2.1 The tower

The tower shape was initially designed to be A-shape. However, harmful out-plane vibration occurred not only when the tower stood alone but also at time of completion of the bridge. Vibration characteristics and mechanism of vibration were analyzed, and the reverse Y-shaped tower was adopted because of its improved aerodynamic properties and aesthetic appearance. As a result of many wind tunnel tests, the corner-cut shape was adopted to the tower section for reducing the amplitude of the vortex-induced oscillations. Cross section measurements are 12 ~ 5.6 m × 8.5 ~ 5.9m, and the tower is the largest class ever of monocell structures.

The overall height of the tower is 220 meters and it consists of blocks divided into 23 levels in height. High-strength friction-grip bolts were adopted to all of the connection in the tower. Two vertical girders were installed in the tower shafts and cast anchor blocks were secured with the bolts to the girders, to which structure the cables are to be anchored.

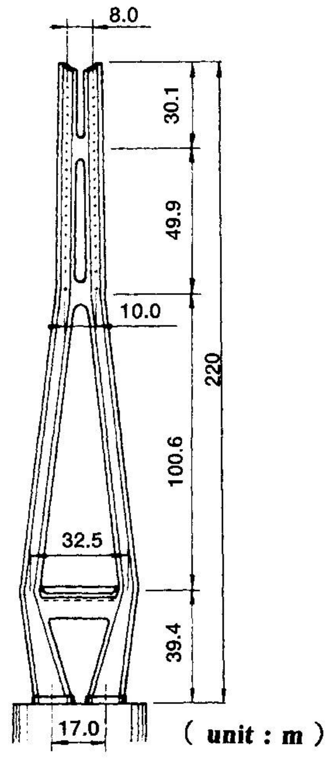

Fig. 2 Front View of Tower

2.2 The cables

Galvanized steel wires of 7mm in diameter were bundled in factories in a semi-parallel form with slight torsion. Then, they were directly coated with extruded high-density polyethylene. With long-spanning, the natural frequency of the cables is remarkably lower compared with those of another cable-stayed bridges (the longest cable: outside diameter 170 mm[ϕ 7 × 379], cable length ≒ 460 meters, natural frequency = 0.225 Hz). For this reason, the properties of rain vibration in the low vibration area were investigated by a wind tunnel test using full-scale cables, and so were the effects of anti-vibration measures by applying incremental damping addition and aerodynamic-oriented cross section. Indent application to the surface of cables in a discrete manner serves as a valid measure since it enhances aerodynamic stability of the cables without substantially exceeding drag force coefficients of conventional cylindrical cables.

2.3 The girder

As for the steel girders, box girder cross sections including fairing were selected for securing aerodynamic stability based on wind tunnel tests. The girders are wide and long flat 3-chamber deck slabs. The girder height is 2.7 meters, and the beam span depth ratio is about 1/330, being of remarkably slender shape.

As a result, the stiffness of the girder was relatively low and axial compressive stress was dominant for determination of the area. Therefore, trough ribs for deck slabs and lower flanges were used, and flat ribs were used for webs. Thus, all of them were designed as compressive stiffened plates.

In consideration of aesthetic and maintenance aspects, the cable-fixing parts were stored inside the fairing so that the fixing structure is not exposed outside of the girders.

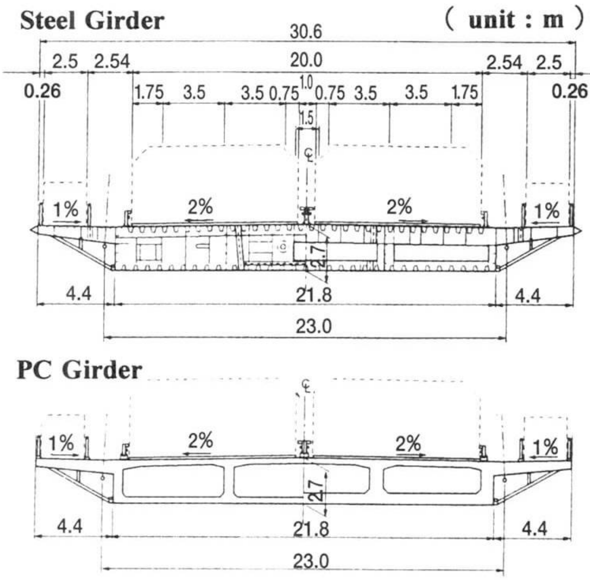

Fig. 3 Cross Section of Girder

In the steel girder manufacture, taking into account that the steel girder is designed as a compressive stiffened plate, careful sizing of members was carried out to minimize initial strain on deck slabs and lower flanges to consequently avoid deterioration in ultimate buckling strength. Based on the results of various fatigue tests of the deck slabs, the following measures were taken: specifying penetration amount of welding trough ribs to deck slabs (about 80% of board thickness), backfilling of scallops of lateral beam webs that trough ribs are installed through, reducing scallop dimensions of the reverse welding part of deck slabs (30 × 75 mm), grinder finishing of close welding which is right below the wheels, etc.

The support condition of the girder is as follows: with an aim to control dispersion of horizontal force and excessive displacement in the longitudinal direction according to earthquakes, elastic support was adopted for the bearing at the tower, and movable bearings were adopted for the other supports. For the elastic support method at the tower part, what was adopted is horizontal shear spring bearing support using non-damping type rubber, which also serves as vertical reaction bearing. The spring value was set at 4,000 t/m/Br.

3. Erection of the superstructure

3.1 Outline of the erection

Erection procedures of the superstructure are shown in Fig. 4 .

The tower is divided into a total of 23 blocks. First, base plates (w ≒ 120t × 2), the first blocks of the tower (w ≒ 240t × 2) and large block of the lower part of the tower (w ≒ 1,500t), were erected by floating cranes. Following, the large block of the steel girder near the tower (2P-side: l=123m, w ≒ 2,000t,

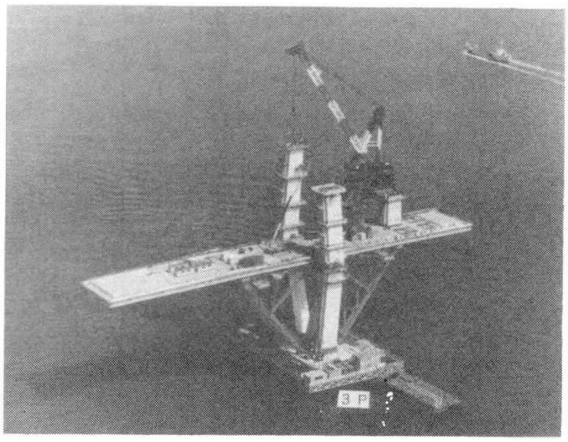

Photo. 1 Erection by Tower Crane

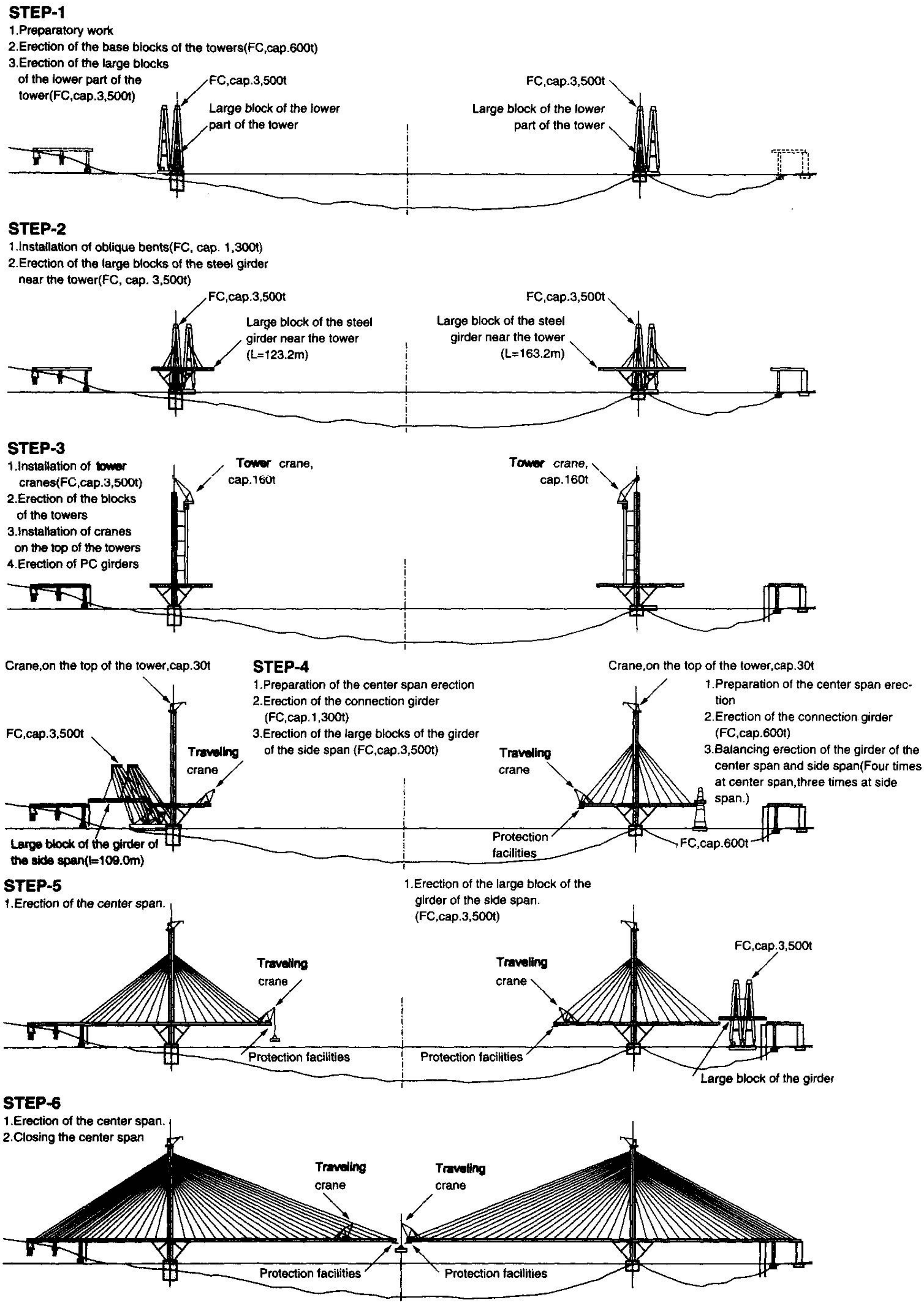

Fig. 4 Erection procedures of the Superstructure

3P-side: l=163m, w ≒ 2,500t) was erected by a floating crane (cap.3,600t). Making use of this large block as the work yard, upper blocks of the tower were erected one by one with a tower crane (lifting cap. 160t) set on the girder. The erection precision of the tower (slant quantity of the tower) was high at the level of around 1/7,000 which was made possible by sufficient manufacturing precision control at factories and careful erecting operations on the site. It successfully met the required erection precision (1/2,000 of the tower height).

The erection unit of the girder can be divided into: the large block deck near the tower, the large block deck at the side spans, and the ordinary block deck (l=20m, w ≒ 300t). Since the side span of the 2P-side is shorter than the 3P-side, large block deck (l=109m, w ≒ 1,800t) is erected at the side span by the floating crane (cap. 3,600t) immediately after the completion of the tower erection. The side span of the 3P-side is longer than the 2P-side, so it was not possible to erect the remaining side span parts as a single unit of the large block deck. While adjusting the balance between the center span and the side span, four blocks were erected at the center span and three blocks at the side span in the following way. Ordinary block deck girders were erected by the traveling crane (cap. 350t) on the girder for the center span, and the floating crane (cap. 600t) was used for the side span. The remaining side span was erected as a large block deck (l=102m, w ≒ 1,500t) by a floating crane (cap. 3,600t).

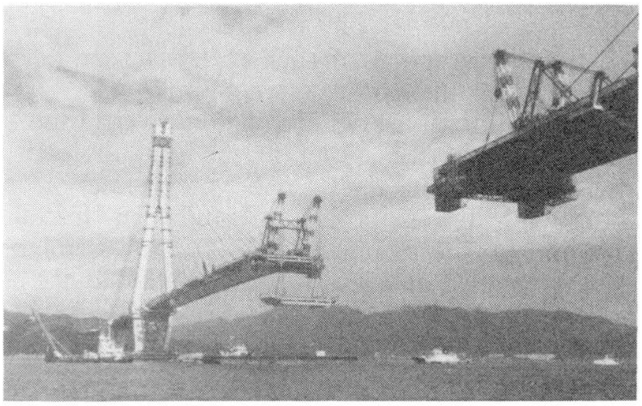

Photo. 2 Erection of Steel Girder

After completing the girder erection of the side span. At the center span part, the cantilever girder erection of ordinary block and cable erection using the traveling crane (lifting cap. 350t) placed on the girder and a truck crane (cap. 360t), were carried out as one process. This process was carried out 18 times for the 2P-side and 15 times for the 3P-side.

3.2 Remarkable features of the erection

Major characteristics of the superstructure erection of the Bridge are: cantilever method of the center span reaching as long as approximately 435 meters maximum due to the center span length of 890 meters, and erection of the girder-suspending cables with the maximum length of 460 metersr.

Regarding the cantilever erection, topographical constraint made it impossible to install temporary bent equipment. Therefore erection was carried out by completely employing the cantilever system. Prior to adopting this method, safety at the time of completion and erection was confirmed not only by examining numerical analyses but also by wind tunnel tests using a whole bridge model on a scale of 1/70.

During the erection of the girder outrigger (in 1997), as many as two typhoons hit the site (Typhoon eight at the end of June, and Typhoon nine at the end of July). Typhoon nine, which came immediately before the closure, especially worried us. However, no damage was caused at all, and the closure was able to be completed, thanks to previously planned countermeasures against typhoons. The countermeasures were: firm binding as anti-vibration measures of cables, retracting the traveling crane to lessen the projected area normal to wind direction, and rolling up the safety nets of the railing. The wind velocity of Typhoon nine was about 25m/sec at the maximum instantaneous wind speed. The relative displacement of the girder at this time was around 50cm.

With regard to the cable erection, pulling-in and anchoring at the girder side were first carried out after anchoring at the tower side. At anchorage fitting at the tower side, the highest cable reaches 170 meters above the girder, and the dead load alone is as heavy as 25t. The capacity of a tower-top crane which was no lift the cables needed to be upgraded, and a crane with a lifting capacity of 30t was necessitated. In pulling-in and anchoring efforts at the girder side, the maximum pulling-in length to the predetermined anchorage point reached about 150 meters. The pulling-in force at this time was as great as 800t maximum. Therefore, pulling-in of the cables to the girder side was divided into three phases: the first pulling-in was done with a winch (pulling-in cap. 30t), the second pulling-in with a wire clamp (cap. 70t), and the third with a center-hole jack (cap. 800t). Thus, cable pulling-in operations were carried out depending on the capacity of each equipment. The tension rod, at the highest cable (l=460m, w ≒ 60t) employed for the third pulling-in operation with the use of the center-hole jack, was ϕ 180mm × 8.5m.

Lastly, field observation of each part has been performed to primarily investigate the impact of winds during the construction of the superstructure of the Bridge. The following were executed: vibration observation of the tower at the time of erection, and vibration observation of the cables and the girder at the time of cantilever erection. After the girder closure limiting the duration up to one year, vibration observation of the cables and girders is on-going with the emphasis on study on spatial correlation of winds and the vibration observation of the cables. The spatial correlation has been conducted with ultrasonic aerovanes installed at five different points that are on longitudinal direction.

4. Postscript

The Bridge venture was decided on in 1989, and the construction ceremony was held in August of the following year, followed by starting the substructure construction in November 1992. The construction moved on to submarine drilling of the tower base, installing caissons, driving installation of underwater concrete and atmospheric concrete. In March 1995, the final driving installation of concrete was carried out, and the construction of the tower base (2P, 3P) was completed. Regarding the superstructure construction, the tower construction was initiated in August 1995. In June 1996, the erection of the final (23^{rd}) block was completed. After a period of preparatory works for erecting girders, erection of an outrigger between the center spans was started. In the summer of 1997, after a lapse of nine years since the venture was launched, the closure of the girder with a center span of 890 meters was observed.

Lastly, we would like to mention that the superstructure of the Tatara Bridge is progressing without any accidents occurring since the start of construction. We would like to express our gratitude to the efforts of those concerned with the construction.

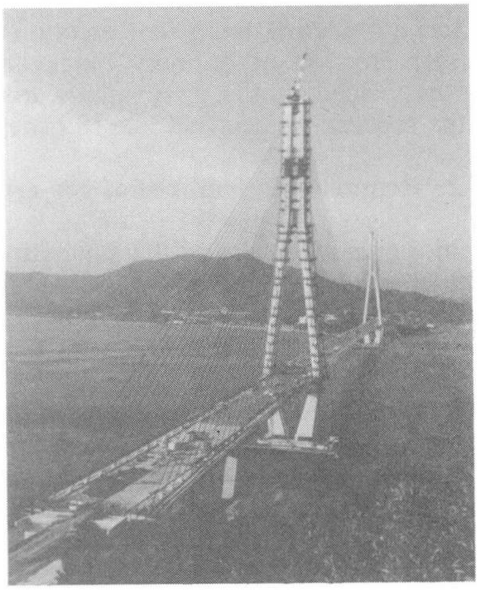

Photo. 3 Tatara Bridge

Future Trans-Strait Road Projects and Technological Issues

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Summary

Creation of new traffic axes for regional revitalization and developments for the coming century requires the construction of super long-span bridges surpassing the Akashi Kaikyo Bridge by 20 to 30 percent. The studies were made so far to realize more economical and rational construction of super long-span bridges. The results have confirmed the possibility of reducing construction cost by developments of new structural types and introduction of new design concepts in place of the construction based on existing technologies. The studies also identified the technological issues to be resolved in the future.

1. Introduction

The Akashi Kaikyo Bridge was completed on April 5th 1998 as a part of the Honshu-Shikoku Bridges project, one of Japan's typical trans-strait road projects. In addition, the Kurushima Bridges and Tatara Bridge are scheduled for completion in the spring of 1999. Then the Honshu and Shikoku will be linked by the three routes as originally planned.

Furthermore, for the 21st century, the plans to develop new traffic axes are proposed in Japan with a view to making effective use of limited nation and building balanced national structure. In addition, a lesson from the Hyogoken-Nanbu Earthquake is the need to create a national structure that allows for certain redundancy. For such backgrounds, trans-strait road projects, as a part of new traffic axes, are being envisaged at such places as the Tokyo Bay Mouth, Ise Bay Mouth, Kitan Strait, Kanmon Strait and so on as shown in Fig.-1.

2. Toward the implementation of future trans-strait road projects

2.1 Need of technological developments

The various technologies were developed for the design and construction of the Akashi Kaikyo Bridge and other bridges on the Honshu-Shikoku Bridges project. The results have enabled safe and reliable construction of long-span bridges having a center span length in the order of 2,000m. Continued and sophisticated use of such technologies are important to the success of future trans-strait road projects.

The meteorological and oceanographic conditions in terms of topography, geology, water depth, wind speed and so on at planned sites in the future projects are, however, expected to be more severer than in the Seto Inland Sea area where the Honshu-Shikoku Bridges were built. Specifically, most future projects involve considerable widths and deep water levels. Therefore the bridge planning requires superstructures with longer spans and substructures appropriate for deeper water. The future projects, facing oceans, are highly vulnerable to typhoons and ocean waves. Some future projects are situated in areas prone to major earthquakes. These conditions may cause super long-span bridges, which are planned in future trans-strait road projects, to be 20 to 30 percent larger than the Akashi Kaikyo Bridge. The technologies used for the construction of the Honshu-Shikoku Bridges can certainly ensure the technological viability of the construction of super long-span bridges surpassing the Akashi Kaikyo Bridge. However, the developments of new technologies are essential to their more economical and rational design and construction.

2.2 Outline of technological developments

At present, the technological issues expected to reduce construction cost and period are being studied intensively as a top priority. The main technological developments are classified into two major categories, namely 1) developments of new structural types and 2) introduction of new design concepts. The technological developments are outlined in Table-1.

Table-1 Outline of technological developments

	Item	Outline of technological developments
Developments of new structural types	Bridge deck cross sections and cable systems with better aerodynamic stability	Slotted box girder and box girder with cross hangers are reviewed to increase aerodynamic stability without increasing dead weight.
	Underwater foundations with better earthquake resistance and economy	Twin-shaft type foundation is reviewed to increase earthquake resistance and economy.
Introduction of new design concepts	Wind resistant design	Fluctuation characteristics of natural wind are reviewed, and the possibility of wind load reduction is proposed. Flutter analysis is developed.
	Earthquake resistant design	Earthquakes considered in designs are defined in two levels, and earthquake resistant designs are proposed for each level.
	Design of superstructures	Safety balance in entire suspension bridge systems including cable safety factor is reviewed. Live loading methods considering actual passing of vehicles are studied.
	Design of substructures	Appropriate methods of evaluating bearing capacity and deformation of ground are studied.

(1) Review of new structural types

(i) Bridge deck cross sections and cable systems with better aerodynamic stability

Wind resistance is one of the most important themes in the design of super long-span bridges. Means of improving their wind resistance include increasing of stiffness of stiffening girders and innovation of bridge deck cross sections. While the stiffening truss girder was used for the Akashi Kaikyo Bridge, the box (one-box) girder was adopted on the Kurushima Bridges because of the center span length of about 1,000m.

Super long-span suspension bridges having a center span length exceeding 2,000m involves such problems as 1) the dead weight increase with enhancement of stiffening girders, and 2) high vulnerability to wind when truss girders are used which have great drag force. Therefore, slotted box (two-box) girders and box (one-box) girders are being examined based on the following assumption. The first is that the box girders can improve aerodynamic stability through enhancement of oscillation and aerodynamic characteristics of bridge deck cross sections. The second is that the hoisting erection method is applicable in which box girder blocks are hoisted directly from sea level right below installation points.

The slotted box girder has an opening at the center of girder (Fig.-2). The relationship between the opening pattern, location, width and other factors, and the critical flutter speed was examined. With the box girder, no critical flutter speed (assumed at around 80m/s) could be secured when the span was longer than 2,000m. Therefore, the improvement of oscillation characteristics by connecting girders and main cables with cross hangers was examined (Fig.-3).

As a result, it was confirmed that the both types were effective to improve the aerodynamic stability. At present, however, reviews are being continued for the slotted box girder. Because the box girder with cross hangers have a problem with the structure of connection between cross hangers and girders. The trial design confirmed that the adoption of the slotted box girder would require approximately 30 percent less weight than existing truss girders. Furthermore the ripple effects on main cables and main towers could make the entire suspension bridge more cost-effective.

(ii) Underwater foundations with better earthquake resistance and economy
The planned sites of the future projects, as compared to those of the Honshu-Shikoku Bridges, have such characteristics as 1) proximity to seismic center of large earthquakes, 2) large water depth at the planned points of their foundations and 3) poor bearing ground. Therefore, the cylindrical solid foundations for main towers, like foundations of the Akashi Kaikyo Bridge, cannot provide satisfactory earthquake resistance and economy.

So that twin-shaft type foundation shown in Fig.-4 is being studied. Some of the benefits of this type are 1) a small amount of concrete required leading to less cost and shorter construction period, 2) light weight applicable to relatively soft ground, and 3) light weight and low center of gravity which increase earthquake resistance. Further the studies are necessary about the structural details of the base of the shaft and the technologies of underwater reinforced concrete, etc. This type foundation, which is spread one, will be applied to individual future projects considering design conditions in terms of water depth and geology at the planned sites of those.

(2) Review of new design concepts
(i) Wind resistant design
As concerns wind resistant design of long-span suspension bridges, the various studies and experiments were practiced in the design of the Honshu-Shikoku Bridges. Besides the improvements were gradually made to increase accuracy. Based on the knowledge obtained in the full model wind tunnel studies, in particular, for examining the aerodynamic stability of the Akashi Kaikyo Bridge, a number of new concepts have been presented for information about future wind resistant design of super long-span bridges.

1) Review of wind fluctuation characteristics
With an increase of span, cross section of girders become more likely to be determined by wind load. More accurate wind load calculation, therefore, would contribute to greater economy. Wind load is calculated by adding an increment value based on wind fluctuation to static load calculated from mean wind speed. As a result of the full model wind tunnel studies at the Akashi Kaikyo Bridge, and of observation of natural wind, it was found that the effects

of wind fluctuation on long-span bridges were relatively small. The studies revealed the possibility of mitigating wind load in the design of long-span bridges, which was lighter than that calculated based on the existing standards.

2) Development of flutter analysis
In wind resistant design, static design against wind load is only insufficient. Measures to prevent flutter phenomenon at the wind speed not exceeding design level are also important. Past wind tunnel studies solely used a section model supported by spring. It was found that with increasing spans, a section model could not fully recreate actual oscillations of long-span bridges. In the wind resistant design for the Akashi Kaikyo Bridge, the full model having a total length of about 40m was adopted in the wind tunnel studies to recreate complex oscillations occurring on the actual bridge and to verify the aerodynamic stability. The results of the studies showed the effectiveness of flutter analysis as a more accurate means of grasping the aerodynamic stability of long-span bridges. For the new structural types such as the slotted box girder, however, the validity of the flutter analysis must be checked by the full model wind tunnel studies.

(ii) Earthquake resistant design
The planned sites of the future projects are close to the epicenters of plate boundary-type earthquakes, and the existence of active faults in the vicinity is also pointed out. In order to practice economical design while ensuring the safety of structures, appropriate input ground motions for design and corresponding analytical methods need to be adopted.

As a basis of earthquake resistant design, the ground motions assumed in the design and the corresponding safety of structures will be considered in two levels. At level-1, "damages which destroy transportation functions shall be prevented for moderate ground motions induced in the earthquakes with high probability to occur within the life time of structures". At level-2, "recoverable functional damages shall be allowed, but collapses shall be prevented for extreme ground motions induced in the earthquakes with low probability to occur". As level-2 earthquakes, two types of ground motions must be considered. The first is the ground motion which could be induced in the plate boundary-type earthquakes. The second is the ground motion developed in earthquakes at very short distance attributable to active faults.

For level-1, earthquake resistant design is made by response spectrum method with the non-linearity of ground. For level-2, check is made by elasto-plastic time history response analysis based on finite element methods. In the future, evaluation method for level-2 ground motion needs to be established.

(iii) Design of superstructures
Of the design standards for superstructures of the Honshu-Shikoku Bridges, those having great potential for reduction of construction cost and period were reviewed.

1) Allowable stress of main cable
On super long-span suspension bridges having a center span length exceeding 2,000m, the weight of main cable accounts for about 20 to 30 percent of total dead weight. Reduction of main cable weight may, therefore, make great contributions to reduction of construction cost and period. During the design of main cable for the Honshu-Shikoku Bridges, allowable stresses of 56kgf/mm2 and 64kgf/mm2 were adopted for the Innoshima Bridge (completed in 1983) and Seto Ohashi Bridge (completed in 1988), respectively. In the design of the Akashi Kaikyo Bridge, an allowable stress of 82kgf/mm2 was used by increasing the tensile strength

from 160kgf/mm2 to 180kgf/mm2 through the improvement of cable strand material.

In the design of super long-span suspension bridges, it was found that an allowable stress of main cable could be set at 100kgf/mm2 on such grounds as 1) sound quality of high-strength strands, 2) high erection accuracy, 3) prospect for enhanced rust-proofing technology and 4) room for further review of safety balance of the entire bridge system. The trial design confirmed that the allowable stress of 100kgf/mm2 in combination with the effects of less new type girder weight could reduce the weight of main cable by about 40 percent.

2) Methods of live loading
In the design of super long-span suspension bridges, live load is basically reduced according to a span as practiced in the design of the Honshu-Shikoku Bridges. In addition, live load is placed only on traffic lanes width unlike conventional loading on carriage way width, in view of the actual passing of vehicles.

In the design of main tower, influence line loading has been practiced in which live load was placed in the range where the load becomes the largest for a designing section. However, there is so little possibility of such loading in reality that it is now possible to increase an allowable stress.

(iv) Design of substructures
The future projects include the foundations which need to be laid on poor bearing grounds. For considering such cases, the studies are being made to grasp the non-linearity of ground based on soil investigations in laboratory using boring samples, and to evaluate the bearing capacity and deformation of ground accurately.

2.3 Technological issues to be resolved

The researches and studies have been made as to the various technological issues with a view to realizing super long-span bridges planned in future trans-strait road projects. As a result, as described above, it was found that the developments of the new technologies could substantially reduce construction cost by about 40 percent as compared to the use of existing technologies for the design and construction of super long-span bridges. The technologies to be developed, however, involve the technological elements, as shown in Table-2, which need to be established in the future. These technological elements will, therefore, be established to ensure the reduction of construction cost, and new technologies and structural types will also be studied which have not yet been given sufficient attention, toward further reduction of construction cost and period.

3. Conclusions

For the construction of super long-span suspension bridges, it was confirmed that the developments of the new structural types and introduction of the new design concepts could enable substantial cost reduction as compared to the use of existing technologies. In the future, the remaining technological issues will be tackled to ensure the reduction of construction cost, and new technologies and structural types will be studied toward further reduction of construction cost and period. For each project, the investigations of natural conditions at the planned sites will be continued and reinforced. Then based on their results, the studies will be made to make the design and construction plans more economical and rational in view of the unique local situation.

Table-2 Major technological issues to be resolved

Item	Technological issues
Wind resistant design	Full model wind tunnel studies for confirming the validity of flutter analysis
Earthquake resistant design	Evaluation method for level-2 ground motion
Design and construction of superstructures	Reviews of design methods which enable labor saving in work, and studies for further reduction of erection period
Design and construction of substructures	Establishment of large-scale underwater reinforced concrete technology Application to individual projects considering each conditions, and studies for reduction of construction period

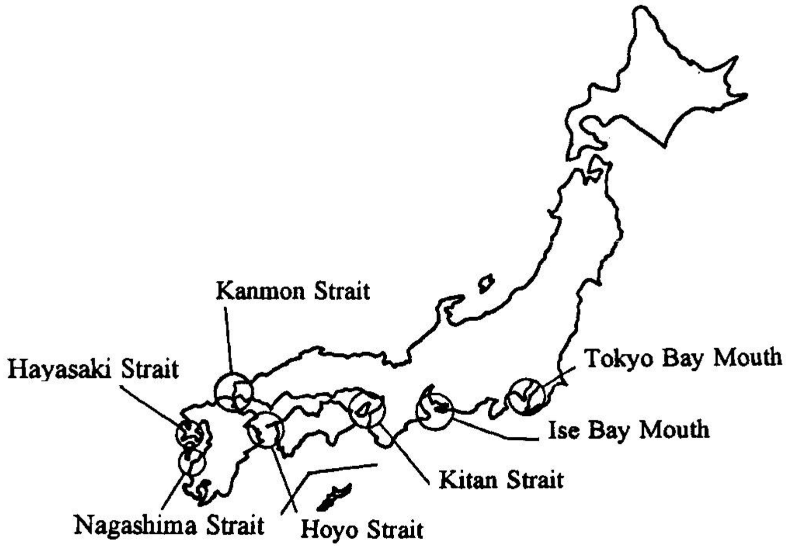

Fig.-1 Major Trans-Strait road projects

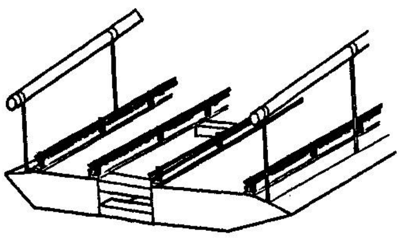

Fig.-2 Slotted box girder

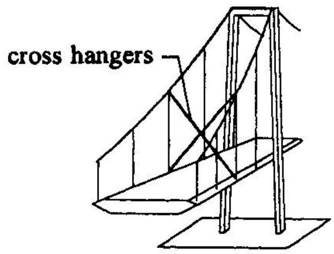

Fig.-3 Box girder with cross hangers

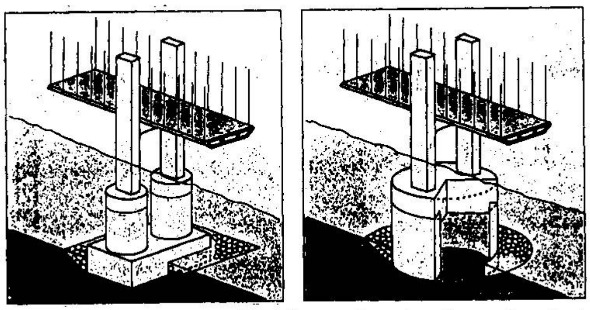

Fig.-4 Twin-shaft type foundation (left) and cylindrical solid foundation (right)

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New Types of Undersea Foundations for Next Generation Projects

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Summary

In Japan, various types of undersea foundations have been developed for use in the construction of the Honshu-Shikoku Bridge and other bridges across straits. But the next generation projects now at the planning stage will require the construction of deep water foundations under conditions more severe than the Honshu-Shikoku Bridge. If these conventional foundations are used under such conditions, their size will be huge, and their construction will be difficult or even impossible. The authors have proposed two new types of undersea foundation: twin tower foundation and hollow rigid foundation. They have also analyzed the settlement of foundations using FEM based on the ground conditions at proposed project. The results have confirmed the stability of foundations with dimensions far smaller than those of conventional foundations. They have also pointed out problems with the construction.

1. Undersea foundations constructed in Japan

Figure 1. shows past executions of deep water foundations in Japan. It presents the relationship between the water depth and the depth of the bottom surface of the foundation for typical types of foundation. The following are the characteristics of each foundation type. The laying-down caisson type is constructed where the bearing layer is close to the sea bottom. Figure 2 shows how the depth of executions of this foundation type has increased. The main tower foundation of the Akashi-Kaikyo Bridge is the largest of this kind; constructed in water that is 50m in water depth, with its bottom surface 60m below the surface of the sea, and consisting of 354,000m^3 of concrete. Caisson foundations are constructed in water that is no more than 15m water depth, but the maximum depth of the foundation bottom surface is 60m. When diaphragm wall foundations are constructed under the water's surface, it is necessary to provide a filled cofferdam. The deepest foundation of this kind that has been constructed is 55m in depth, but it will be able to construct to a depth of more than 100m because it is excavated with machinery. Multi-column foundations are executed at water depth ranging from 5m to 20m. The maximum pile embedment depth of this type is 70m, using concrete piles with a diameter of 10m.

The above facts indicate that the principal factors that determine the type of undersea foundation are the depth of water, the depth from the sea bottom to the bearing layer, and the scale of the load. Where the bearing layer is shallow, a laying-down caisson is best regardless of the depth of water. Where the bearing layer is deep, consideration is given to the use of a column type foundation such as caisson or diaphragm wall foundation and to a pile type foundation such as a multi-column foundation or bell type foundation. Because the former types provide high rigidity, they are useful where the load will be large, while the latter are best where the water is deep.

2. Proposal of new type foundation

Figure 3 shows the locations and outlines of the ocean strait where highway bridge projects now at the planning stage. The routes, bridge types, and foundation locations for each projects are all under study at this time. The following are the hypothetical foundation conditions. The water depth at the main tower foundation ranges from 20m to 70m. The bearing layer of the ground is softer than that under the Honshu-Shikoku Bridge.

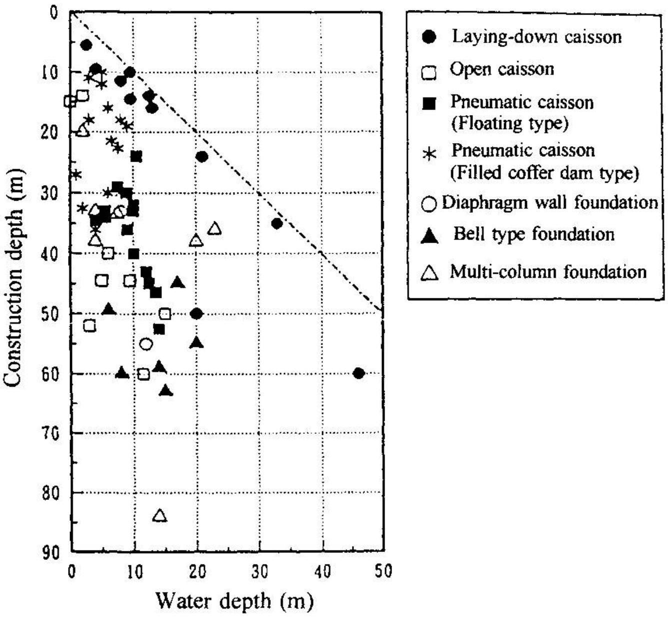


Fig. 1 Completed deep water foundation executions

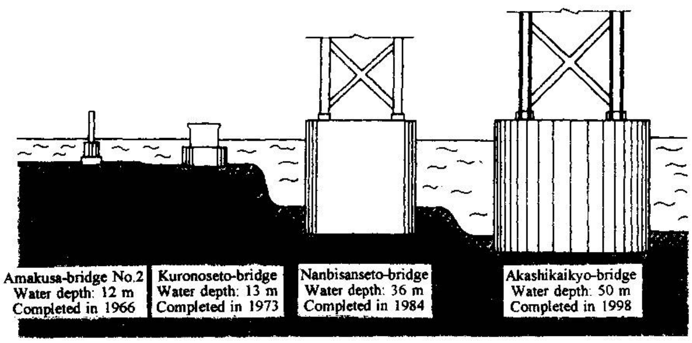
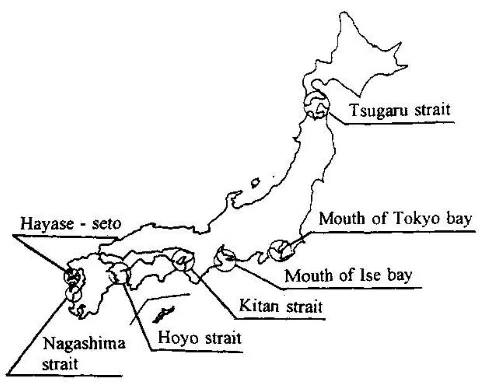


Fig. 2 Completed laying-down caissons

The anchorages are generally planned installed on land or on a beach, but depending on the project, some are at a depth of nearly 50m from the ground surface to the bearing layer and others are about 40m under the sea surface.

Trial calculations have been performed in order to study the limits of the water depth where it is possible to employ the laying-down caissons, and to devise measures to expand this range. For the trial calculations, the load of the superstructure was calculated assuming a suspension bridge as large as the Akashi-Kaikyo Bridge. The results are a vertical load of 130,000tf, a horizontal load of 3,800tf (corresponding to horizontal seismic intensity of 0.2), and moment of 380,000tf-m. The water depth were assumed to be 50m and 80m. The allowable bearing capacity of



Strait name	Distance from shore to shore	Maximum water depth
Tsugaru strait	East side : 13 km	270 m
	West side: 19 km	140 m
Mouth of Tokyo bay	15 km	80 m
Mouth of Ise bay	20 km	100 m
Kitan strait	11 km	120 m
Hoyo strait	14 km	200 m
Hayase - seto	5 km	120 m
Nagashima strait	2 km	70 m

the ground during an earthquake was assumed to be 150 tf/m^2. Past laying-down caisson foundations have all been cube shaped or column shaped with a 100% solid section. The deeper the water, the greater the dead load of a foundation, and possibly, a rise in the required section dimensions. For this reason, as shown in Figure 4, trial calculations were performed for 50% hollow section cubes and narrow top inverted T sections in addition to 100% solid section cubes. Figure 5 shows the foundations' plane dimension - maximum subgrade reaction relationships. If a 100% solid section cube foundation is installed where the water is 50m deep, the foundation's dead load will be high so that even if its plane dimensions are increased, the subgrade reaction will not be reduced very much. In order that the subgrade reaction be lower than the allowable bearing capacity, the plane dimensions must be 175m x 175m, which is unrealistic. In the case of a cube with a 50% solid section or an inverted T shaped foundation, the subgrade reaction force is lower than the allowable bearing capacity at plane dimensions of 60m x 70m.

As the results of the above trial calculations, where the water is deep, the foundation section should be reduced in order to lower the dead load of the foundation body. Based on these results, the authors have proposed the twin tower foundation and hollow rigid foundation shown in Figure 6 as improved versions of the older solid rigid type. The twin tower foundation is one that lowers the main tower itself directly to the sea bottom. In addition to the reduction effect of the concrete volume of the foundation body, it will provide superior seismic stability because its center of gravity is low and its inertia during an earthquake is small. Because the weight of this foundation is low in proportion to the area of its bottom surface, it can even be used on ground that is not hard. And because the area exposed to tidal currents and wave forces is small, it is very stable at normal times. A hollow rigid foundation is a conventional solid rigid type with a hollow space formed inside it. Where the bearing layer is good quality, it is possible to reduce the bottom slab of the foundation as shown in the figure, and it is also possible to sharply reduce the finished surface area of the sea bottom surface excavation.

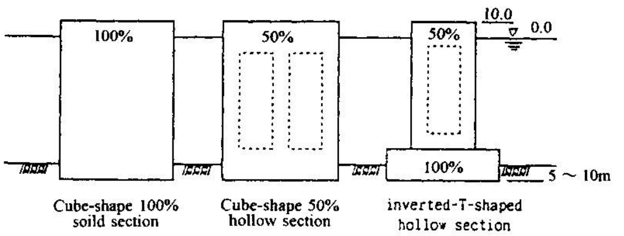
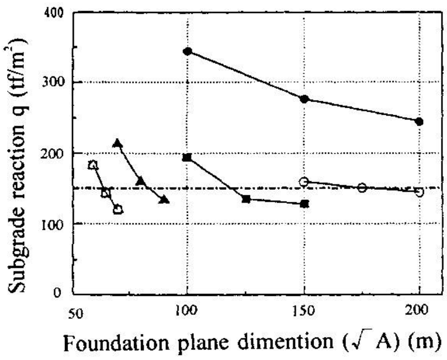
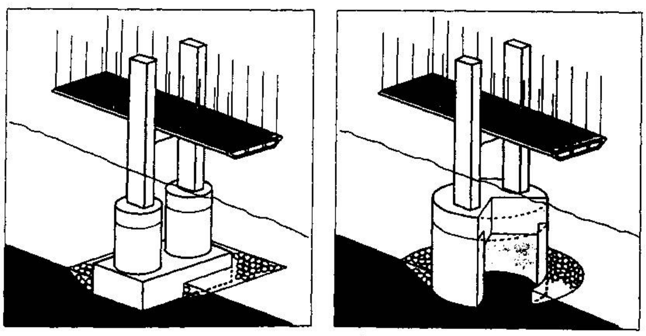


Fig. 4 Laying-down caisson used for the trial calculation



Type	Water depth 50m	Water depth 80m
Cube 100%		
Cube 50%		
Inverted-T-shape		

Fig. 5 Dimensions and subgrade reaction of a foundation



(a) Twin tower foundation (b) Hollow rigid foundation

Fig. 6 Proposed new foundations

3. Foundation design method

The selection of the bearing layer is an important factor to determine the construction costs. Because it is dependent to a large degree on the quantity of soil to be excavated to reach the bearing layer as well as on the dimensions of the foundation. In order to create a rational design in such cases, the bearing capacity and settlement of the foundation must be calculated with high precision and the results reflected in the design.

The twin tower foundation and hollow rigid foundation can both be categorized as shallow rigid foundations for design purposes. The bearing capacity and deformation calculation method of shallow foundations used to design ordinary bridges would present the following problems.
[1] The shear strength of the ground used in bearing capacity calculations is evaluated lower instead of ignoring its confining pressure dependency and strain level dependency.
[2] Because the modulus of deformation of the ground used in deformation calculations is a value obtained from plate loading test etc., the strain level of loading test does not conform with the strain in actual bridge foundations, and the deformation of the foundation is over-estimated.

In order to design a foundation with greater precision than in the past, it is necessary to use a calculation method that can account for these problems. One method under consideration would involve precisely modeling the physical properties of the ground at the site based on geological exploration and laboratory testing, and performing FEM analysis incorporating these properties to forecast the quantity of deformation of the foundation. The following is the specific procedure that would be used to do this.

[1] Clarification of the geological structure at the planned foundation location by carrying out boring, sonic prospecting and seismic velocity logging at the site.
[2] Using undisturbed specimens obtained by boring for precision laboratory testing to clarify the deformation properties of the ground including everything from the minute strain level to the failure range.
[3] Based on the results of [1] and [2], formulating the deformation properties of the ground to be used for numerical analysis by considering the confining pressure dependency and the strain level dependency.
[4] Performing calculations to determine the dimensions, load conditions, etc. of the foundation.
[5] Performing the numerical analysis in sequence according to the construction procedure in order to calculate the behavior of the foundation.

When the settlement of the Akashi-Kaikyo Bridge foundation was analyzed based on this procedure, the analytical and measured values coincided closely, confirming that the method is an effective way to forecast the settlement of a large foundation.

Next, an example of a trial calculation of a deep water main tower foundation performed with this method is described. The bridge used in the trial calculation is the suspension bridge shown in Figure 7. The water depth is 43m and the bearing layer is 10m below the sea bottom. The plane dimensions of the foundation were determined based on the dimensions necessary to fix the tower base to the foundation as well as on the space of two legs of the main tower. Figure 8 presents the results of the settlement forecast analysis of this foundation. The results of the analysis show that the foundation would subside very little, less than 4cm, under the load imposed when the bridge is completed, which is

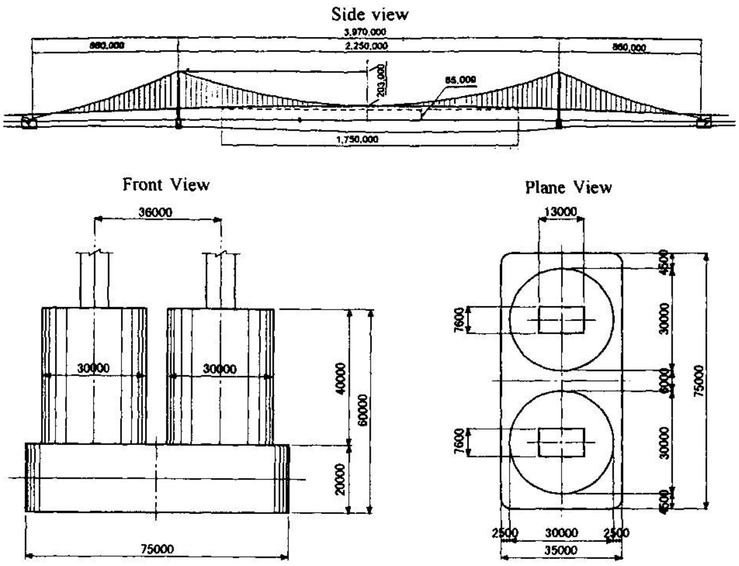


Fig. 7 Profile of bridge

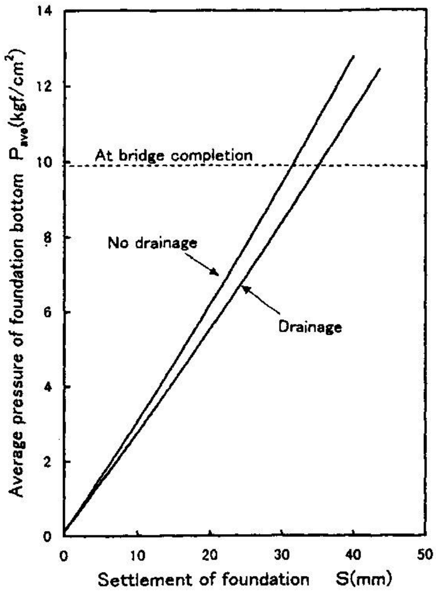


Fig. 8 Main tower foundation settlement forecast analysis

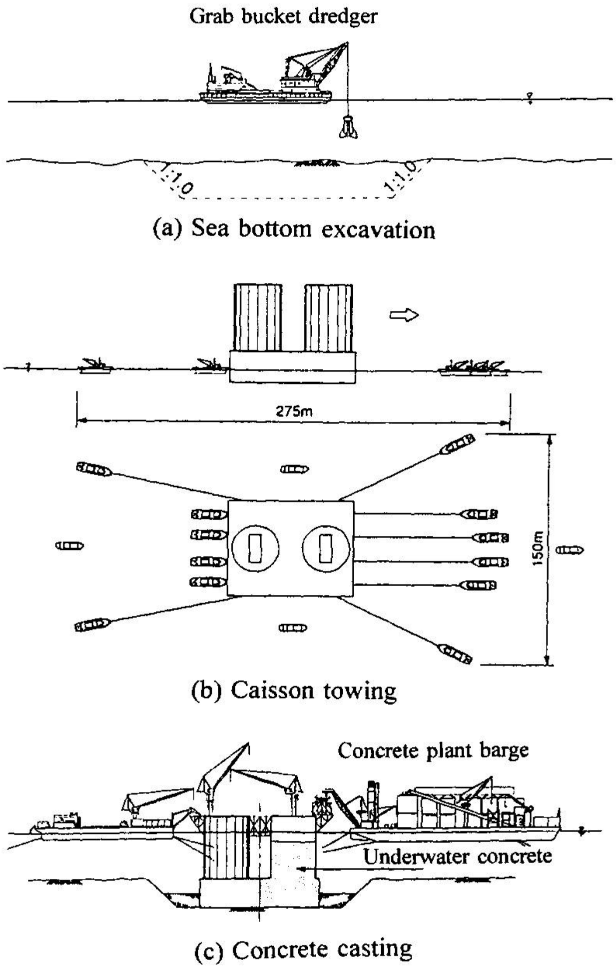


Fig. 9 Foundation execution procedure

little, less than 4cm, under the load imposed when the bridge is completed, which is sufficient for a suspension bridge foundation. The dimensions of this foundation would be far smaller than those of a conventional solid foundation, which would permit sharp reductions in both construction costs and the construction period. And a separate study of the stability of a foundation with these dimensions during an earthquake has confirmed that it satisfies the required safety level.

4. Foundation construction method

The construction methods for the new foundations proposed above were studied. The following paragraphs outline the problems studied in order to improve the construction method and construction efficiency, with the twin tower foundation used as an example.

[1] Sea bottom excavation (Figure 9 (a))
A grab bucket dredger is used to excavate the bottom down to the bearing layer. To shorten the construction period, the finishing surface area is reduced. When the

volume of excavation increases because the bearing layer is deep, consideration is given to discharging the excavated soil with an efficient pump boat. Because the work is performed in the ocean facing the open sea, a study is performed to select the type of boat that will guarantee a good working ratio.

[2] Fabricating and towing the caisson (Figure 9 (b))
After the caisson is fabricated at a dock, it is towed on the ocean surface to the foundation location. As a twin tower foundation, it is not as stable during towing as conventional foundations, but it is possible to guarantee stability with supplementary stabilizing techniques.

[3] Laying the caisson
After the towing operation is completed, mooring lines are connected to the caisson to adjust its location, then water is injected and it is installed at the fixed location.

[4] Foot protection
Foot protection is installed around the caisson by a clamshell dredger as a measure to prevent scouring, increase the resistance of the caisson's front surface, and to prevent mortar leakage.

[5] Casting concrete (Figure 9 (c))
After the interior of the caisson has been cleaned, underwater concrete is cast. A concrete plant barge is used to prepare the concrete. In order to provide underwater reinforced concrete, it is necessary to develop a casting method that permits the concrete to fill every part of deep water large cross section area foundations. As in the case of task [1]sea bottom excavation, the operating rate of the plant barge is improved.

[6] Main tower anchor frame installation
A crane barge is used to install the main tower anchor frame.

5. Conclusions

In order to realize the next generation projects that are now at the planning stage, it is necessary to construct deep water foundations under harsher conditions than those encountered at the site of the Honshu-Shikoku Bridge. To meet this need, deep water foundations constructed in Japan were surveyed, and a trial design was performed to study the application water depth limit and measures to raise this limit. The twin tower foundation and hollow rigid foundation were proposed as new types of foundations that will realize these projects now at the planning stage, and methods of design and construction of these two new foundations were studied.

The new foundations proposed in this report are improved versions of conventional foundations, so a radical new foundation concept will be a prerequisite to any further construction cost reductions. Furthermore, the above study looked at next generation projects for which specific studies have been undertaken and the deepest site in this group is about 70m. But projects that have not progressed beyond the conceptual stage include some that would face even more severe conditions, and in such cases, it will be necessary to expand preliminary studies to include, for example, the jacket type foundation.

In conclusion, the authors would like to express their sincere gratitude to the members of the Strait Highway Project Technology Committee that has already studied the technologies presented above.

Prevention of Thermal Cracking in the Anchorage of the Akashi Kaikyo Bridge

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Summary

The Akashi Kaikyou Bridge will be the longest bridge in the world in 1998 after completion, which length is 3,990m and center span is 1,990m. The understructures consist of a couple of anchorage and foundation of towers. A large volume of concrete , which is approx. 140,000m^3, is needed for the each anchorage (1A and 4A) to resist the designed force. Construction method of such massive concrete is usually constrained by thermal stress due to heat of hydration causing severe cracking. In addition to the volume, rapid construction was required. Several new methods have been developed and applied to fulfil the requirements, such as low heat cement, pre-cooling, pipe cooling and highly workable concrete. The process of applying these new methods and results of the construction were described in this paper.

1. Procedure of selecting construction method of 1A

Remarkable requirements of constructing 1A anchorage of the Akashi Kaikyo Bridge are to solve problems of the cracking and rapid construction, which has to place approx.140,000m^3 of concrete during 28month including setting anchor-frame and many steel structures. Since the horizontal area is 5500m^2 and the height is 47.5m, whole volume of concrete can't be placed at once and it has to be divided into several blocks and lifts. Area of a block was decided to take the length of the anchor-frame into account, and height of a lift was planed to take thermal crack index and total period of concreting into account. Planned area of blocks and lift schedule are shown in Fig.1 and 2. Method of preventing cracking was designed based on "the design standard on cracking due to thermal stress for

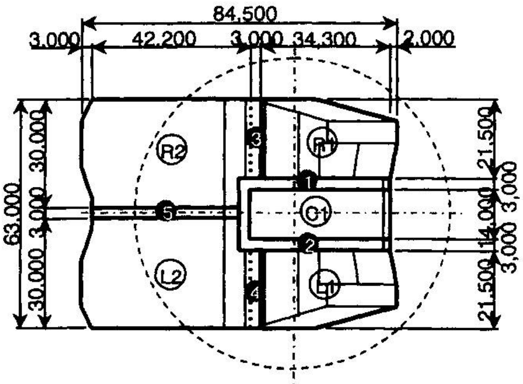

Fig.1 Planned Blocks for Construction

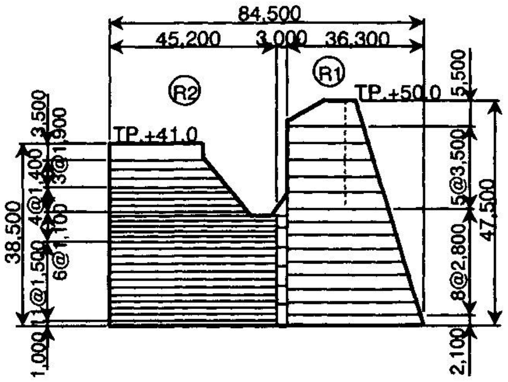

Fig.2 Planned Lifts for Construction

massive concrete in bridges", which was authorized by Honshu Shikoku bridge authority. Thermal stress was calculated by FEM for candidates of construction methods and thermal crack index (strength/calculated thermal stress) was assessed for each case. The most suitable method by considering the value and economic effects were decided to fulfil the above value. The lowest designed thermal crack index was 1.2, which means to allow small cracking but to prevent severe one. Concrete temperature, cooling method, and height of a lift were planned to fulfil this limit.

2. Construction method of 1A for prevention of cracking

The following methods were adopted for actual construction to prevent cracking.

1) Low heat cement, in which large portion of Portland cement was replaced to possolan, such as fly ash and blast furnace slag, was developed and applied. Adiabatic rising temperature of the concrete with 260kg/m^3 of cement was less than 25℃. Calcium carbonate was adopted as powder to achieve self-filling property without heat generation.
2) The new type of concrete mixing plant with two mixers which capacity were each 6m^3 was constructed, where water chiller and ice plant making flake ice were attached.
3) Pipe cooling by chilled water and pre-cooling by flaked ice was adopted for whole concrete and placing temperature of all concrete was less than 20 ℃.
4) Special pre-cooling with liquid nitrogen was temporary adapted to support the above mentioned pre-cooling. It was applied for upper part of concrete placed in summer where 15℃ was required due to excessive height to 3m.
5) Since 3m width of slot were stetted at the side of next block, individual concreting schedule could be planed without constraint of the next block. It decreased the period of concreteing.

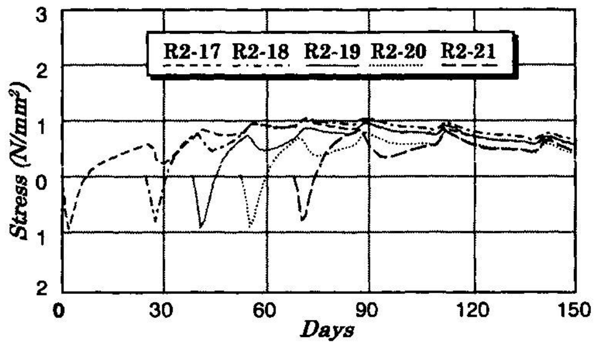

Fig.3 Measured Stress

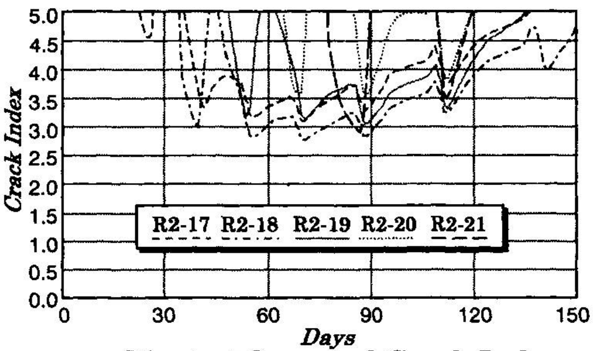

Fig.4 Measured Crack Index

3. Results of the construction

The stress and crack index in the center of continuous several lifts, which were concreted in high atmospheric temperature, was shown in Fig.3,4. The temperature was measured less than 38 ℃ and the maximum generated tensile stress was about 1.0 N/mm^2. This indicated that little risk of cracking was existed during construction. After the temperature reached 16℃ 3m widths of slot were finally filled with concrete, and the concreting work was completely finished until 1996.4. Both rapid construction and control of cracking were satisfied in this construction of 1A anchorage.

Construction of the Kurushima Bridge

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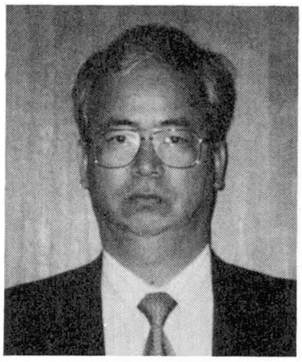

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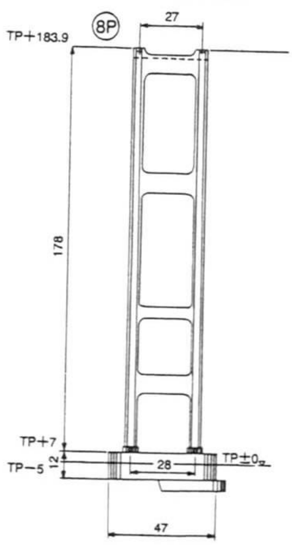
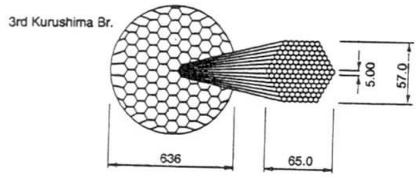
1. Outline

The Kurushima Bridge, located on the Onomichi-Imabari Route of the Honshu-Shikoku Bridges, is the three connecting suspension bridges across the Kurushima Strait. The Kurushima Strait with its beautiful scenery is designated as one of Japan's national parks. The strait is approx. 4 km wide and divided into three marine traffic routes by the two small islands. Compounding the heavy shipping traffic, the strait is hazardous for its geographical conditions characterized by numerous

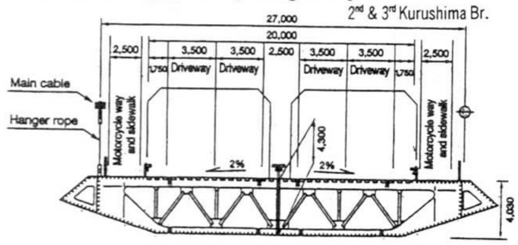
● Tower Front View



● Cable Section



● Stiffening girder [box girder] Section



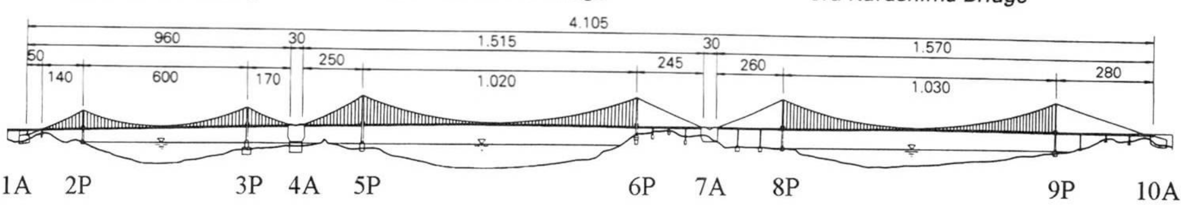


Fig. 1 General view of the Kurushima Bridge

islets and swift tidal current with a maximum velocity of 10 knots. The construction work started in September of 1990 and will be finished in 1999.

2. Towers

The tower height is generally determined so that the sag ratio may become optimum in dynamics and economy and the line linking the tower tops may become parallel to the vertical alignment of the girder. However, in the case of this bridges, this manner brings about the discontinuity of the towers' height, because the center span length of 1st bridge is shorter than that of 2nd and 3rd bridge. Therefore, the special consideration was paid for aesthetics and the towers' heights were determined in the manner of continuous variation through the comparative study.

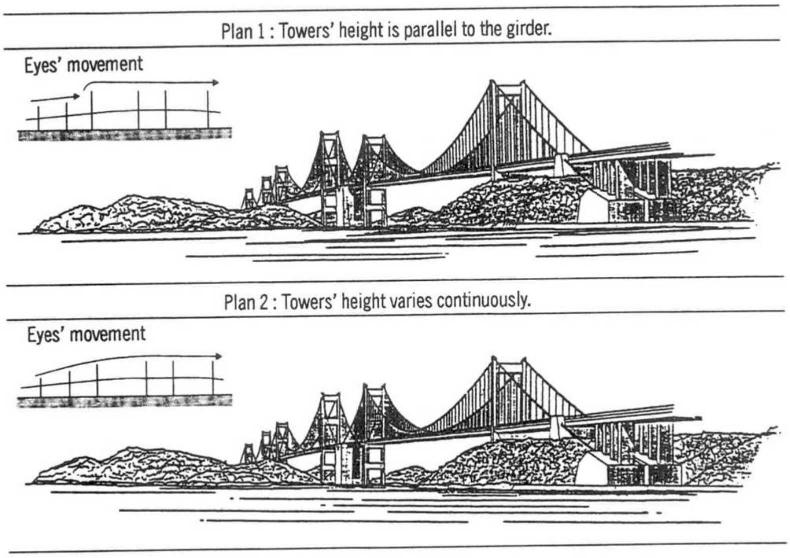


Fig.2 Comparison of looks in different tower heights

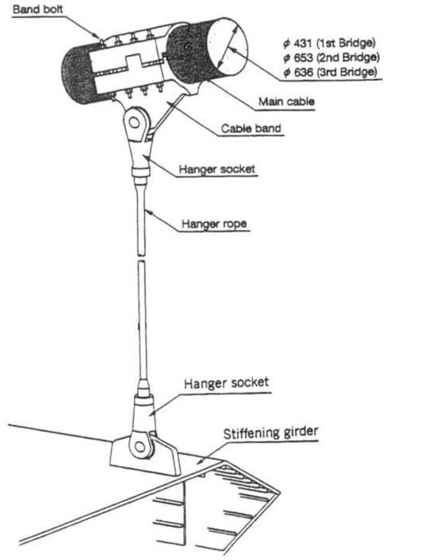


Fig.3 Hanger rope fixing structure

3. Cables

Cables are wired with the prefabricated-strand method. Every strand is comprised of 127 recently developed high-strength steel galvanized wires of 180 kgf/mm2 which were also adopted for the Akashi Kaikyo Bridge. Each hanger rope is a bundle of 5 mm-diameter galvanized steel wires of high-strength grade arranged in parallel. The rope surface is covered with polyethylene for corrosion protection of the hanger rope, to which fluorocarbon resin coating is further applied for coloring. The hanger rope is connected both to the main cable and the girder with pin.

4. Girders

Stiffening girder is erected by the perpendicular hanger method. A prefabricated-at-factory girder blocks on a barge is placed under the perpendicular location of the erection firstly, and it is lifted to fix to the hangers by the lifting machine on the cables. In order to stand the strong tidal current, a sailing barge, was originally developed to secure a certain position without mooring any anchor.

Fig.4 View of the girder erection

Full Model Wind Tunnel Test of the Akashi Kaikyo Bridge

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Summary

The aerodynamic stability of the Akashi-Kaikyo Bridge was evaluated through wind tunnel tests using full model. The test results were compared with analytical results, and there were new findings in the flutter characteristics and gust response characteristics, e.g. contribution of drag of unsteady aerodynamic force to flutter, effect of spatial correlation of turbulence on gust responses, etc.

1. Introduction

In the design of a long-span bridge, aerodynamic stability is a very important item. In the case of Akashi-Kaikyo bridge, a suspension bridge with main span length of 1990.8m, wind tunnel tests using full model were conducted as well as section model test. The geometric scale of the model was 1/100. The wind tunnel tests using the 40m long model (Picture-1) were conducted at a wind tunnel which has a test section of 41m wide, 30m long and 3m high.

2. Flutter Characteristics

In smooth flow, the damping of torsional mode becomes negative at the wind speed of 8.4 m/s (Fig.1). In the vibrational shape during flutter (Fig.2), vertical component was not negligibly small and was complicated. This means the flutter observed was coupled flutter and multiple vertical bending modes contributed to.

The results of first 3-dimensional flutter analysis, where moment and lift of unsteady aerodynamic forces due to torsional and vertical vibration were considered, did not agree with test results (see Fig.1). The result of second analysis, where all the unsteady aerodynamic forces were considered, agreed with test results (see Fig.1). From parametric study, drag of unsteady aerodynamic force due to vertical and torsional vibration was found to be effective.

3. Gust Responses Characteristics

As an example of observed gust response in turbulent flow, horizontal component at the center of center span when the intensity of turbulence was 9.6% is shown in Fig.3.

The results of conventional gust responses analysis did not agree with test results well (Fig.3). From parametric study, major reason of the difference between test results and analytical result were found to be as followed including other components.

① Horizontal component: difference in spatial correlation of turbulence between measured and used in the analysis.

② Torsional component: aerodynamic damping which was usually neglected in the analysis.

③ Vertical component: difference in spatial correlation and aerodynamic admittance.

The result of the second analysis where above mentioned factors were considered, agreed with test results fairly well (Fig.3).

4. Closing Remarks

Through wind tunnel tests using full model of the Akashi-Kaikyo Bridge, various aerodynamic characteristics of long span bridge was found.

Photo.1 *Full Model of the Akashi-Kaikyo Bridge*

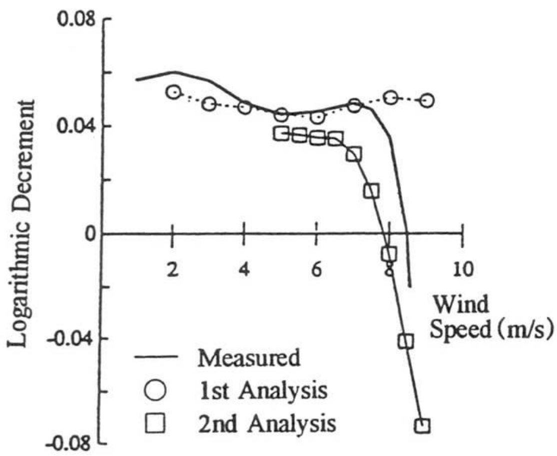

Fig.1 Damping of the Model

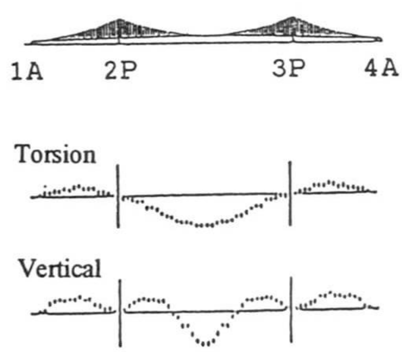

Fig.2 Vibrational Shape during Flutter

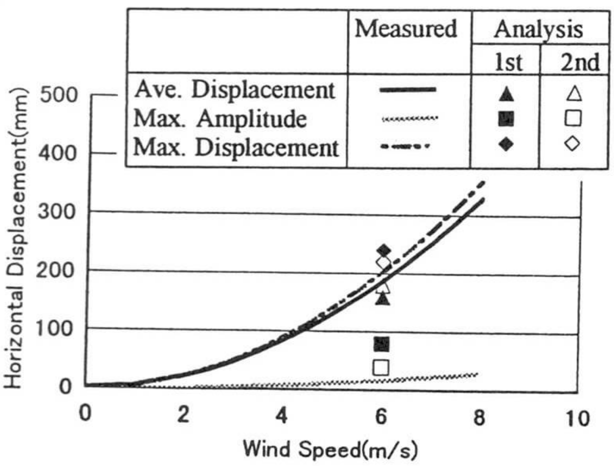

Fig.3 Horizontal Gust Responses

Results of Monitoring and Simulation Analyses for Deep Excavation

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Summary

1A anchorage foundation of the Akashi-Kaikyo Bridge had the deepest excavation whose scale was 64m depth and 80.6m inner-diameter, which had not experienced before. And, there was also some concern about bottom ground failure due to the groundwater pressure. Accordingly, we monitored behavior of the retaining wall as well as condition of bottom ground by using more than 500 instruments. And also we estimated wall deflection and ground movement by simulation analysis in order to predict and check the stability of them during the excavation. The paper describes results of monitoring of the retaining wall and the bottom ground and simulation analyses in this excavation work.

1. Abstract of deep excavation work

The excavation work of 1A anchorage foundation of Akashi-Kaikyo Bridge has characteristics as follows.

① The excavation work inside a cylindrical earth retaining wall is larger by 20% in diameter and deeper by 40% than any excavation in the past that Japan had experienced before.

② The ground water level (TP±0m) is high to the final excavation level(TP-61m).

③ The slurry wall for retaining and cut-off wall is not penetrated into impermeable layer. Overall-view of excavating condition is shown in fig.1.

2. Results of monitoring and simulation analyses

2.1 Displacement and concrete compressive stress of earth-retaining wall

The displacement distribution of the retaining wall is shown in fig.2. The maximum displacement was measured at final excavation stage. The measured value was 1.2~1.5cm, although the design value was 1.8cm. While, the maximum of circumferential concrete compressive stress was generated at same stage too. The measured value was 9MPa, although the design value was 13.5MPa. The reason why measured value showed two third of the design value was concluded that external force acting on the wall was smaller than design value. (see fig.3)

2.2 Heaving and hydro-fracturing of the bottom ground

We evaluated permeability of Kobe stratum correctly by using measured results of groundwater pressure and simulation with 3 dimensional FEM seepage flow analysis.

Fig.4 shows the accuracy of simulation analysis of groundwater pressure at final excavation stage. It is found that the simulated value is very similar to the measured value. The occurrence of hydro-fracturing was evaluated by local-safety factor, which used Mohr-coulomb's failure criterion. Fig.5 shows distribution of the local safety factors in the bottom ground. The minimum value was 1.24. The excavation work was judged to be in safety by these analyses before the excavation as having been shown later..

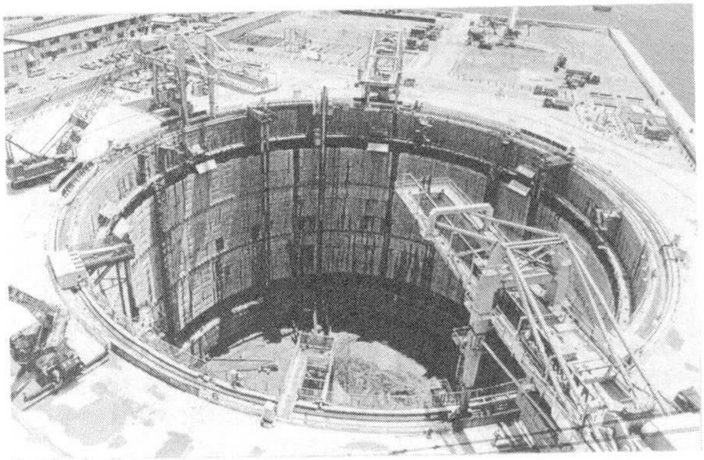

Fig1:Overall-view of excavating condition (Current depth of excavation = TP-54m)

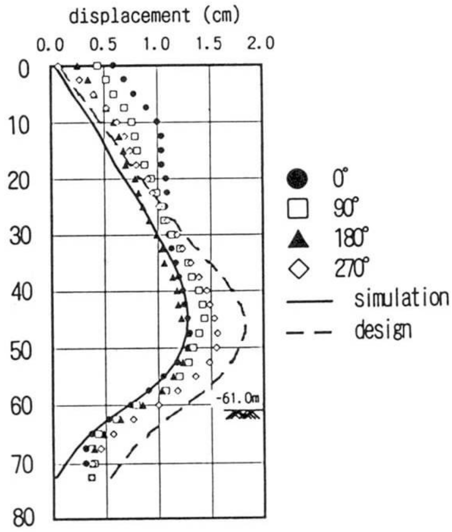

Fig2:Horizontal displacement of earth retaining wall

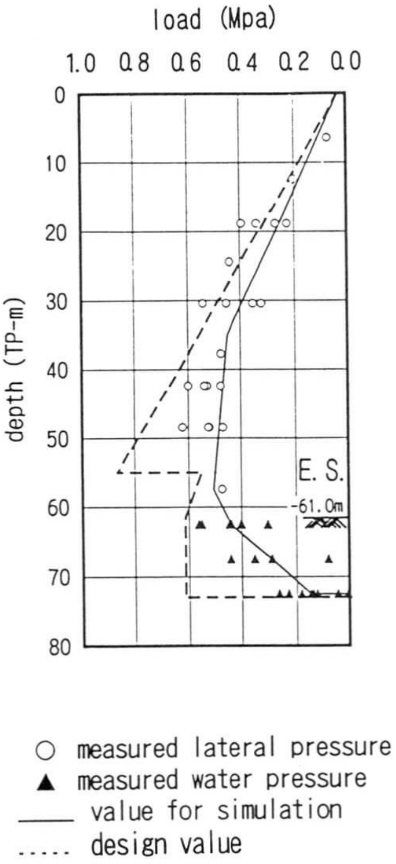

Fig3.:External load acting retaining wall

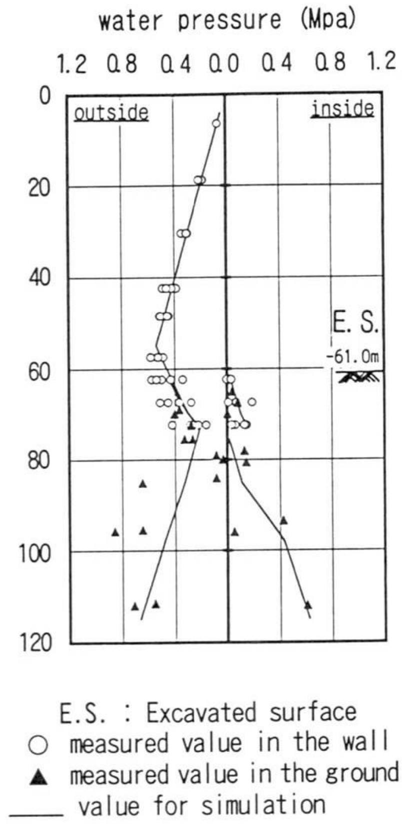

Fig4.:Groundwater pressure distribution

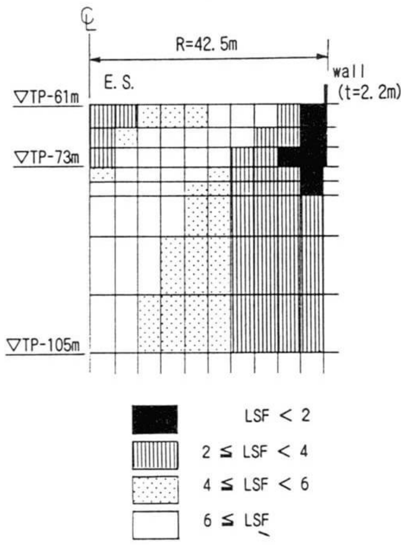

Fig5.:Local safety factor distribution in the bottom ground