Construction of Akashi-Kaikyo Bridge West Anchorage

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Abstract: The Akashi-Kaikyo suspension bridge, with a central span of 1,990 m, will soon become the world's longest single span. In total, some 1,420,000 m^3 of concrete will go into two piers and two anchorage substructures. Two steel towers will rise to 297 m above sea level; 200,000 t of steel will form the superstructure. Twin cables, each 1.1 m in diameter, will support up to 118,000 tons. Design criteria should enable the bridge to survive earthquakes of magnitude 8.5 and winds of over 80 m/s. Started in 1988, it is scheduled for completion in 1998 at a cost of $3 billion. This paper begins with an overview of the bridge, but concentrates on construction of the west anchorage (1A), recently completed. This anchorage was built within a 75.5-m deep, 2.2-m thick concrete slurry wall. Mass concrete within the 85-m diameter retaining wall was placed from a depth of 64.5 m using roller-compacted concrete. Design details, production methods, automated instrumentation, and data-collection systems are described.

INTRODUCTION

The widely known effort to link four major islands in the Japanese archipelago has been under way for more than five decades. One effort is the construction of the Honshu-Shikoku Bridges over the Seto Inland Sea, which separates Honshu and Shikoku (Ishiyama 1988; Mori 1988; Ohashi 1988; *Outline* 1989; Paulson 1981). This enormous system, which got underway in the mid-1970s after years of study, consists of some 17 major bridges plus connecting highways, railways, and tunnels. The entire Honshu-Shikoku Bridge System is composed of three routes, among which the central route, the Seto Great Bridges (formerly called the Kojima-Sakaide Route), was completed in April 1988, and was put in service since then (*Honshu* 1989; Miyoshi 1988). Construction of the two remaining routes is in progress in order to complete the entire Honshu-Shikoku Bridge System by the end of this century. The scale of this system is so immense that, when completed, eight of its bridges will be among the world's 20 longest spans, displacing such well-known structures as the San Francisco-Oakland Bay Bridge from that elite list. In its scale, variety of structures, and level of bridge technology applied throughout, the Honshu-Shikoku Bridge System in unprecedented.

In 1991 terms, the Honshu-Shikoku Bridge Project is slated to cost approximately ¥3 trillion ($24 billion at ¥125 per $1), not counting interest. The budget is justified partly on the highways and partly on the railways. The main method of financing for the highway is bonds and loans from public and private sources, but investment has been made by the central

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government and ten regional governments whose jurisdictions are the primary beneficiaries. The purpose of investment is to lessen the burden of paying interest, and the actual debt cost of the entire Honshu-Shikoku Bridge Project thus becomes 4.8%. The benefits of the land links will include more rapid and reliable alternatives to the weather-vulnerable sea links that were previously used, and opening not only Shikoku but many smaller islands in the Seto Inland Sea to economic development and closer ties to the mainstream of the Japanese economy.

Regardless of location, a project of this scale involves an enormous environmental impact. This particular location happens to be the Seto Inland Sea, which simultaneously is a national park of great scenic beauty and cultural value, a rich fishing ground for a nation where fish is a primary food source, and a transportation network carrying literally thousands of ships per day. In keeping with the scale and importance of the project, enormous efforts have been made to mitigate the potentially negative social, economic, and environmental impacts, and indeed to turn the bridges themselves into scenic attractions that complement their surroundings.

The Akashi-Kaikyo Bridge, which will have the world's longest single span, is one such bridge, and is now being constructed between Kobe-City and Awaji-Island (see Fig. 1). It has been under construction since 1988 and is scheduled for completion in 1998 (Akashi 1990; Omachi 1988; Paulson 1991). In 1991 terms, the Akashi-Kaikyo Bridge Project is expected to cost approximately ¥380 billion (approximately $3 billion). This paper will describe general aspects of the Akashi-Kaikyo Bridge, then focus on the unique construction of the foundation for Anchorage 1A (Kobe-side anchorage), which was recently completed under the direction of the first two writers.

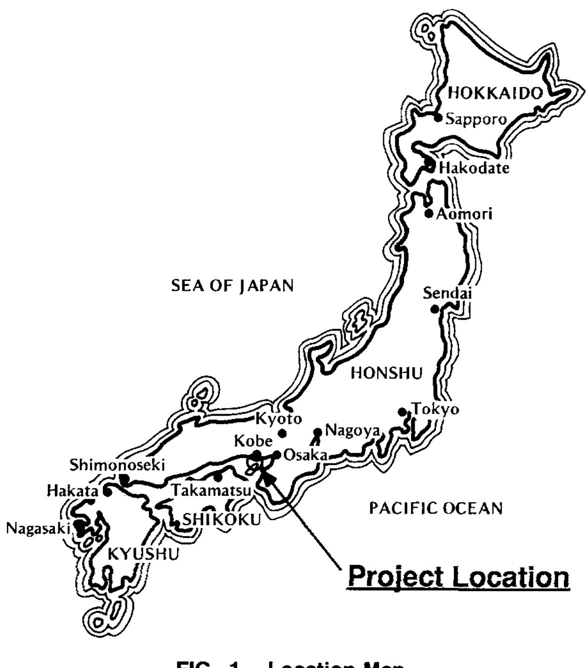

FIG. 1. Location Map

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GENERAL ASPECTS OF THE AKASHI-KAIKYO BRIDGE PROJECT

The Akashi-Kaikyo Bridge is located on the Honshu side of the Kobe-Naruto route, the easternmost of the three routes in Japan's Honshu-Shikoku Bridge System. The Akashi-Kaikyo Bridge is to become the world's longest single bridge span of any type; at nearly two kilometers pier to pier it surpasses Britain's Humber Bridge by a margin of over 40% and will be the crown jewel of the system. The bridge, as shown in Figs. 2–3, will cross over the 4-km-wide Akashi Straits, which has been designated as an international waterway. It is a three-span stiffening truss suspension bridge for a six-lane expressway. It will have a central span of 1990 m (6,529 ft) and two side spans each 960 m (3,150 ft) long, for a total length of 3,910 m (12,829 ft). The design criteria should enable the bridge to survive earthquakes of magnitude 8.5 (Richter), winds of over 80 m/s (180 mph) (Akiyama 1988), and the piers will resist scour from twice-daily tidal currents of up to 4 m/s (8 knots).

The two main steel towers, which sustain the parabolic shape of the cables, will rise to a height of 297 m (975 ft) above sea level because the sag /span ratio of $1/10$ yields the most advantageous design for the entire superstructure even in the case of this gigantic suspension bridge. The steel stiffening truss, which carries the deck for the six-lane expressway, is hinged at each tower, making three main segments. An approximate total of 200,000 tons of steel will go into the superstructure. The twin cables, which support such a huge span, are each 1.13 m (3.6 ft) in diameter, and each is made up of 37,000 steel wires. The maximum horizontal cable pull, against which the two anchorages must resist, will reach 118,000 t of tension when the bridge is completed and in operation.

Erection of the superstructure is now underway. The design has already been subject to extensive seismic and wind-tunnel testing. It is expected that construction will use similar fabrication precision and advanced, fatigue-resistant welding quality standards that were proven on other Honshu-Shikoku structures already completed, and will be worth watching in the years ahead.

The completed substructures consist of two anchorages and two main piers, which are all huge reinforced concrete structures. In total, some 1,420,000 m^3 (1,860,000 cu yd) of concrete will go into the piers and anchorages. Table 1 shows details of their dimensions and volumes. The main characteristics of these foundations are:

Except for good rock at 4A (Awaji-side anchorage), the bearing layer is relatively soft rock (for 1A and 3P) or a diluvial sand/gravel layer (for 2P). The bearing layer is rather deep.

Reaction from the superstructure is large, as aforementioned.

Aseismic design is required because the bridge is constructed in an earthquake-prone area.

For these reasons, the size of foundations inevitably becomes huge.

As for the construction method, the "laying-down caisson method," which was also used in Kojima-Sakaide Route, was applied for the two main piers. The two anchorages, on the other hand, are constructed on reclaimed land as a working base, consequently their foundation work method was decided to be a direct foundation with open-cut excavation.

Heat-of-hydration cracking in concrete has been a technical problem for such mass-concrete structures. A low-heat-generating type of cement, which contains a large amount of finely grained blast-furnace slag (and sometimes fly ash), thus has recently been developed in Japan. The adiabatic temper-

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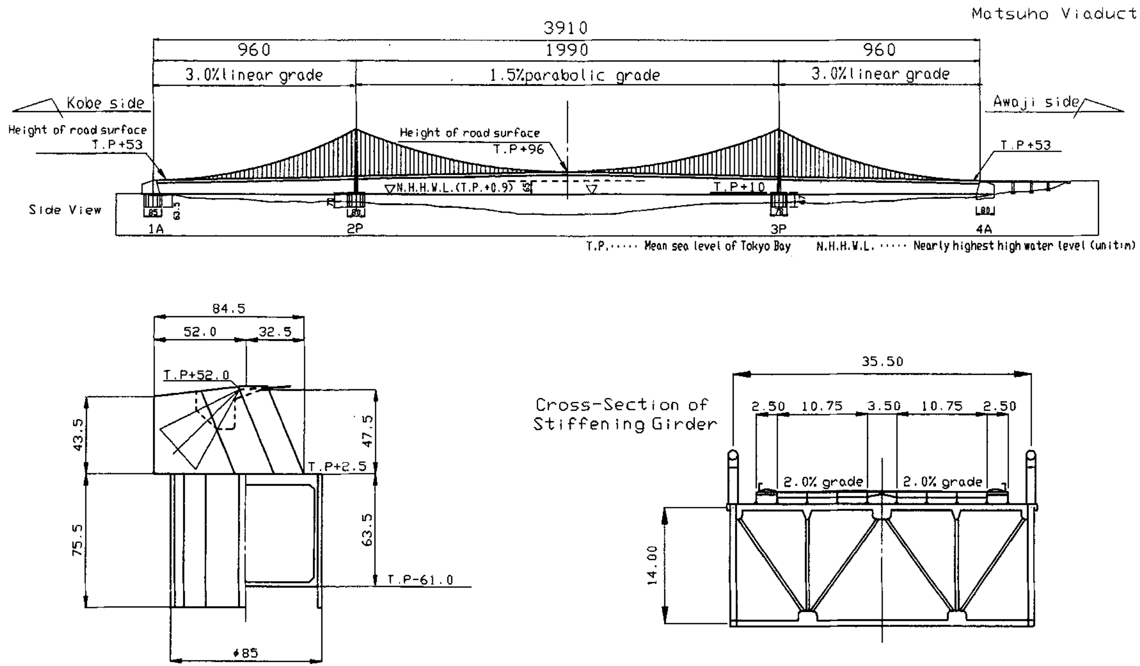

FIG. 2. Site Plan and Elevation of Akashi-Kaikyo Bridge

FIG. 3. Rendering of Akashi-Kaikyo Bridge when Completed

ature rise of such concrete, which is made of such cement of 260 kg/m^3 as the unit content, can be kept below 25°C, and the possibility of cracks can be reduced accordingly. This new type of the cement has been widely used in the Akashi-Kaikyo Bridge Project.

The piers under each of the two main towers sit on foundations some 60 m (200 ft) below sea level, are about 80 m (260 ft) in diameter, and are constructed from caissons floated out, sunk, and each filled with about 322,000 m^3 and 354,000 m^3 of concrete, respectively. Starting with the pier foundations, advanced sensing and control systems were used to monitor and control seabed excavation by the clamshell dredge with its 200-t grab bucket. Because the Akashi Kaikyo piers were set in softer material, large mats of bagged rocky debris were placed around the bottom edge to resist scour from the fast-moving tides. While similar piers on earlier bridges used prepacked aggregate filled with cement mortar from an adjoining batching barge through tremie pipes preinstalled in their caissons, the Honshu-Shikoku builders have sufficiently advanced the formulation of concrete mixtures to be both fluid enough to fill voids and viscous enough to remain undiluted when placed underwater, so large concrete-production barges placed concrete directly underwater within the caissons on this project.

Anchorage 4A on Awaji Island is an open-cut earth and rock excavation some 80 m long by 63 m wide by 26 m deep. It will contain a mass-concrete structure of 94,000 m^3. By any standard it would be a large project in its own right, but not compared to its counterpart on the Kobe side. Thus we will now concentrate on Anchorage 1A.

ANCHORAGE 1A FOUNDATION AND ITS CONSTRUCTION

Anchorage 1A's large cylindrical foundation, located on the Honshu side, is particularly notable for being built within a 75.5-m (250 ft) deep, 2.2-m

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TABLE 1. Main Dimensions of Foundations of Akashi-Kaikyo Bridge

Item (1)	1A (2)	2P (3)	3P (4)	4A (5)
(*a*) Natural Condition				
Water depth	Reclamation land	44–47 m	36–39 m	Reclamation land
Maximum tide	—	3–3.5 m/s	3.5–4 m/s	—
Bearing layer	Kobe-layer sedimentary rock in Miocene	Akashi-layer diluvial sand/gravel	Kobe-layer sedimentary rock in Miocene	Granite
(*b*) Foundation				
Type	Direct foundation	Direct foundation	Direct foundation	Direct foundation
Shape	Cylindrical	Cylindrical	Cylindrical	Cylindrical
Dimension	ϕ85 m × H63.5 m	ϕ80 m × H70 m	ϕ78 m × H67 m	L80 × B63 × D26 m
Bottom depth	−61 m	−60 m	−57 m	−23– −12 m
Construction method	Open excavation with continuous underground wall	Laying-down caisson	Laying-down caisson	Open excavation with piles wall
(*c*) Body				
Water foundation	—	265,000 m³	238,000 m³	L83 × B63 × H53 m
(*d*) Concrete Quantity				
Air foundation	375,000 m³	89,000 m³	84,000 m³	94,000 m³
Water foundation	—	265,000 m³	238,000 m³	—
Body	140,000 m³	—	—	140,000 m³
Total	515,000 m³	354,000 m³	322,000 m³	234,000 m³
(*e*) Amount of Steel				
Total	15,000 t	21,000 t	19,000 t	13,000 t

Note: The grand total of concrete = 1,425,000 m³; steel = 68,000 t.

(7 ft) thick concrete retaining wall constructed using a slurry-trench method. Excavation for the slurry wall was significant because, at a depth of 75 m (246 ft), the excavating machines' sensors and computer systems were able to control tolerances within about 5 cm (2 in.), which is remarkable for a blind excavation under this depth of slurry mud. The mass-concrete backfill within the 85-m (270-ft) diameter retaining wall was placed from a depth of 64.5 m (212 ft) below the ground surface using the roller-compacted concrete (RCC) method. Design aspects, production methods, automated instrumentation, and data collection will be described herein.

The outer diameter of 85 m was decided from design calculations, and the bottom elevation depth of 64.5 m was governed by the geological condition that its bearing layer, called the "Kobe layer," appeared at such a depth, as shown in Fig. 4.

Strata above this could not be the bearing layer for the huge anchorage,

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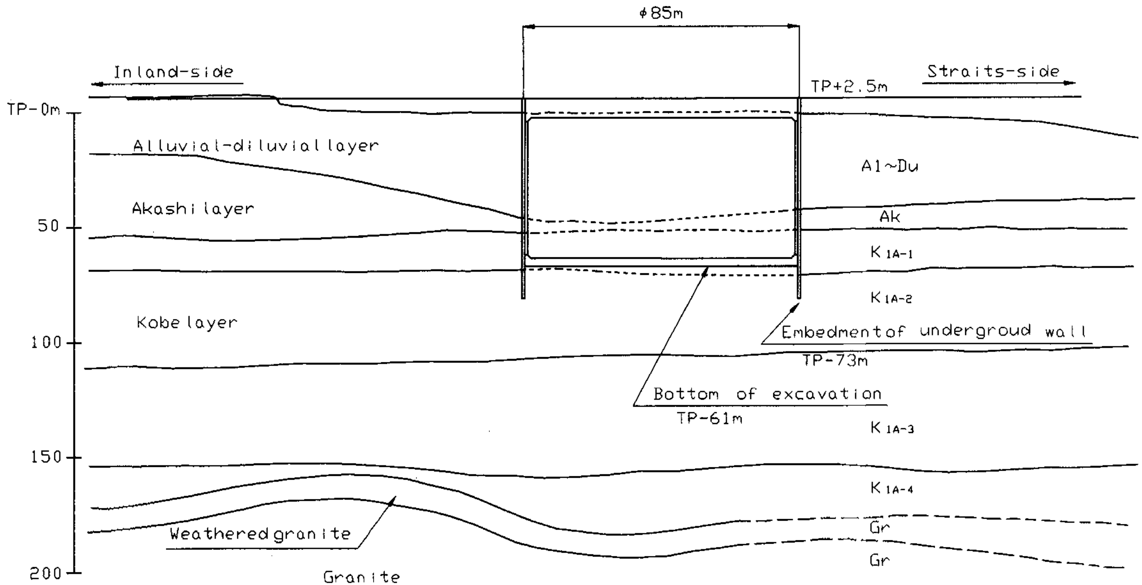

FIG. 4. Geological Conditions

but would have become an obstacle in two ways to sinking the foundation if the foundation had been constructed with a caisson method (either open caisson or pneumatic caisson). The first obstacle was that the surface of reclaimed land could not support the huge cutting edge of a caisson without some kind of temporary foundation work. The second was that an intermediate layer may have exerted too much friction against sinking. It was thus decided that the foundation should be constructed with the following method.

1. Reclamation of the working yard.
2. Execution of a continuous underground wall by slurry-trench method.
3. Excavation of inside material while utilizing the underground wall as a retaining wall against water/earth pressure.
4. After excavation, the inside is filled mainly with RCC to complete the foundation.

This construction method is an application of a technique developed for the construction of underground storage tanks for crude oil or liquefied natural gas. Many tanks have been constructed in Japan with the same method since the energy crisis in 1973 in order to increase the country's reserves of energy sources.

From a structural viewpoint, this foundation is a so-called tin can type in which relatively thin "reinforced concrete" members surround the huge bulk of inner RCC, which has no reinforcement. Namely, the nature of this foundation is near to a gravitational structure mainly because of the required depth to reach the bearing layer.

CONTINUOUS UNDERGROUND SLURRY-WALL WORK

This section describes characteristics of the underground slurry wall, its design and that of the inner liner wall, and the construction methods that were employed.

Characteristics of Underground Wall Applied in Project

The underground wall used in Anchorage 1A has the following characteristics:

The wall has large thickness (2.2 m) and embedment depth (75.5 m), and its arrangement was decided to be cylindrical to raise its efficiency as a retaining wall during excavation.

The wall is considered to be a part of the main foundation in the design of the whole substructure.

The exact shape of the wall was determined to be a 92-sided polygon in order to make the shape similar to a true circle.

The junction between panels was designed as a so-called concrete touch, in which the edge of concrete of the initially placed (forwarding) panels is cut during the excavation of following in-fill panels so that sound and strong concrete of the two adjacent panels may tightly touch to transmit compressive forces through the panels. To achieve this, precise taper-cutting of the forwarding panels' concrete edges was required.

Concrete for the panels was decided to be high strength so as to reduce its required thickness. Heat of hydration of such high-strength concrete was lessened through the use of low-heat-generating type cement.

In the case of the 1A excavation, an additional 2-m-thick inner liner wall

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was constructed along the underground slurry wall as a further retaining wall, because the scale of inside excavation was tremendously large and it was thus judged that the safety of the whole retaining system should be increased.

Design of Underground Wall and Inner Wall

The retaining-wall system (underground wall plus inner wall) for the 1A excavation had been larger than any previous big excavation in Japan in terms of area and depth. Various technical investigations and comparisons were thus made before finalizing design of the retaining-wall system.

Analysis Model and Method

The underground wall and inner wall were modeled using a two-dimensional (2D) skeleton model and an axial-symmetrical finite-element method (FEM) model. The former model was used for analysis of uniformly distributed water/earth pressure, in which the progress of the work (excavation and construction of the inner wall as well as the ring effect of the retaining system were considered). The latter model was for nonuniformly distributed pressure. Intensity of the nonuniform load was decided to be 20% of the uniformly distributed pressure. In addition to these loads, such effects as the difference of the outer shape of the retaining wall (true circle and 92-sided polygon) and construction errors were considered. Also, the output of sectional force from each model was superimposed for design of the retaining-wall system.

Because the excavation is very large in its scale and the minimum distance between the underground wall and the straits is less than 20 m, potential instability of the excavated ground was judged to pose serious problems. Such items were therefore investigated as a safety margin against heaving, hydraulic fracture, and so on and it was concluded that the stability could be secured mainly with a decrease of underground water pressure. For this, further utilization of deep wells, whose original purpose was to keep the excavation level dry, was judged necessary.

The geology at the site is composed of an alluvial-diluvial layer, called the Akashi layer (a diluvial sand/gravel, layer which is also called the Osaka stratum) and the Kobe layer. The Kobe layer is an alternating sedimentary rock consisting of sandstone and mudstone, which were deposited more than 15,000,000 years ago, and is prone to have creep failure, slaking, and expansion due to absorbing moisture.

Required Thickness and Strength of Concrete for Underground Wall

There is a tradeoff between the required thickness and the strength of the concrete for the underground wall. Here, the design strength of concrete was determined to be 36.3 MPa from various design standards in Japan. For the wall concrete, mix specifications were 20 mm maximum coarse aggregate size, 8 cm slump, and 4% ± 1% air entrainment. The unit cement content reached 430 kg/m^3, and thus use of the low-heat generating type cement was effective. It had been known that the stress distribution from the underground slurry wall to the inner liner wall was not so good, but the thickness (2.0 m) of the latter was determined so as to resist the circumferential compression on its own. This thickness enabled the inner wall work to be done easily.

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Vertical Reinforcement of Underground Wall

The underground wall acts in the vertical direction as a beam supported with the ring-spring, which is introduced to express the shape of the underground wall as being cylindrical. It may be understandable that the wall has no necessity to be stiffened in the circumferential direction where axial compression due to ring effort is dominant, but the wall is reinforced in the vertical direction.

Construction of Underground Wall

Panel Arrangement

The entire underground wall, which was 85 m in outer diameter, 2.2 m in thickness, and 75.5 m in excavation depth, was divided into 23 forwarding panels and the same number of following panels, as shown in Fig. 5. Excavation of the former, which were wider, was done with three continuous cuts, and the center line was bent by 3.92° at each cut so as to better approximate the cylinder shape. The following panels were excavated with one cut while the excavation included the taper cutting of concrete of the already-set forwarding panels. After all the forwarding panels were completed, drilling for following panels began.

Drilling Machine

Drilling machines used at the site belong to the horizontal multiaxial type. The 196-t machines can excavate up to 170 m in depth, 1.5–3.2 m in thick-

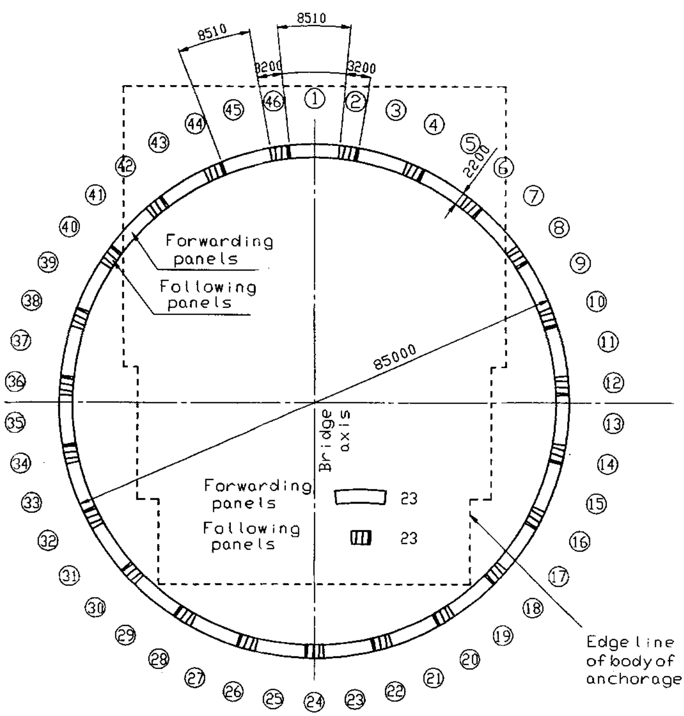

FIG. 5. Panel Arrangement

FIG. 6. Overview of Slurry Wall Excavation on Anchorage 1A

ness, and up to 3.2 m wide in one pass. Cutters rotate from 11 to 16.5 rpm. Their slurry pumps handle up to 10 m^3 per min. The models used here were two Hydrofraise-10000s and one Electro Mill-320, both of which were adjusted to drill 3.2 m × 2.2 m in a single cut. Fig. 6 shows the entire view of the site during the excavation and construction of the wall.

Slurry

To secure the stability of the trench during drilling, slurry containing polymer, bentonite, dispersion agent, and so on always filled the trench. This slurry had another function to transport excavated material up to the ground surface.

Control of Drilling Accuracy

It is very important to maintain drilling accuracy in any continuous underground-wall work, thus the positional accuracy of the center and the verticality of the drilling was always controlled with a computer-assisted system. This was done during the drilling using an ultrasonic device to measure the shape of the trench, depending on the necessity for further controlling the accuracy. Specified allowable error of the center of this work was 50 mm at a depth of 75 m.

Concreting

Construction of 1A required huge volumes of various types of concrete. Accordingly, a large, highly automated on-site concrete plant, whose performance is shown in Table 2, was set up. As for concrete of the wall, total volume was 44,200 m^3, and each placement for the forwarding panel and following panel reached 1,400 m^3 and 510 m^3, respectively. The concrete was strongly made of a ternary blended low-heat-generating type of cement.

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TABLE 2. Specifications of On-Site Concrete-Batch Plant

Item (1)	Capability (2)
Practical daily output	1,800–1,900 m^3/day
Mixer	2-axes forced mixing: 6 m^3 × 2
(*a*) Material storage (Equal to 3 days)	
Cement silo	1,500 t × 1
Fine-aggregate silo	1,000 m^3 × 4
Coarse-aggregate silo	1,130 m^3 × 4
(*b*) Precooling equipment	
Flake-ice production	60 t/day
Ditto storage	100 t
Water chiller	180,000 kcal × 2

Actual strength, confirmed when core samples were taken from the wall, reached about 60 MPa.

INSIDE EXCAVATION

As already partially described, the inner wall was placed along the underground wall to raise the stiffness of the entire retaining-wall system. Because the work area was narrow and deep, both works were carried out separately. This section will describe the overall process.

Alternating Excavation and Inner Wall Work

Excavation proceeded for some depth, then construction of the inner wall for this section followed while the next layer of excavation was held up. During such pauses, computer simulations and estimations to confirm the safety of the next step of the work were done. In this manner, the entire excavation from +2.5 m to −61 m was smoothly and safely executed in five main stages.

Monitoring System

To record such items as stress and deformation of the retaining wall, distribution of the earth and water pressure, and displacement of the bedrock, a monitoring system was set up as shown in Fig. 7, which was composed of many sensors and computers. The placement and the number of sensors are shown in Figs. 8 and Table 3, respectively.

An on-site computer gathered data every 15 min, stored, and processed them to let site engineers understand the situation in real time. The data were also transmitted to the contractor's technical laboratory through public telephone lines for more detailed analyses, which included simulation of the current situation and planning and estimation of the following work steps.

Result of Monitoring and Its Evaluation

Stress, Deformation, and Pressure of Underground Wall

Fig. 9 shows the vertical distribution of the circumferential compressive stress, the vertical bending stress in the underground wall, and ground-water

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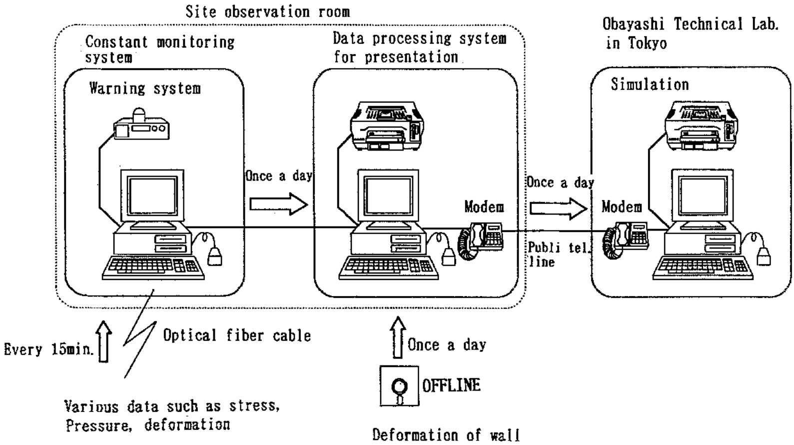

FIG. 7. Monitoring System

pressure at both the initial stage (on January 10, 1991, when the excavation level was -17 m) and the final stage (on August 16, 1991, when the excavation reached -61 m). It is obvious that the circumferential compressive stress (here measured in the rebars) increased along with the progress of the excavation. The maximum circumferential compression was calculated to be about 2,000 t/m^2 from this value instead of design value of 3,000 t/m^2. The main reason for this difference is that lateral pressure acting on the wall was smaller than the design pressure, one example of which is shown in Fig. 9.

The reasons why the water-pressure distribution in the deep portion dropped from the static water pressure are assumed to be that a sound impermeable stratum was encountered at the approximate level of -50 m and that the underground water had been pumped up through four sets of deep well systems. In addition to the decrease of the underground water pressure, it was confirmed that the earth pressure was also smaller than the design value. For these reasons, the underground wall received less compressive force than what the design anticipated. The fact that the underground water pressure dropped significantly also contributed to a rise in the stability of the excavated ground.

Vertical bending stress measured in the rebars shows relatively small values but frequent changes of sign. This is because the entire cross section of the underground wall could be considered effective and that the bending moment changes its sign depending on the location. The former assumption that the underground concrete wall might not have cracks rests on the fact that the actual strength of concrete reached the level of 60 MPa and the tensile strength was accordingly expected to be 6 MPa, which was larger than the assumed tensile stress.

Fig. 10 shows deflection of the underground wall every 90°. It can be concluded that the wall had beautifully symmetrical deformation, and the correlation to the simulation was good.

Displacement of the Ground

Because a large amount of the sand and gravel were removed, the bedrock in the deep portion might have upward displacement. Also, this would

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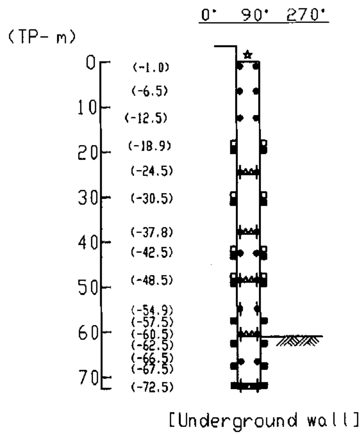

T.P.······Mean sea level of Tokyo Bay

Notes

Mark	Sensors
●	Re-bar stress meter (horizontal)
|	Ditto (vertical)
□	Earth pressure gauge
■	Groundwater pressure gauge
△	Concrete stress meter (horizontal)
▼	Ditto (vertical)
☆	Increnometer
○	Deep ground water pressure gauge
✱	Sliding type dislacement type

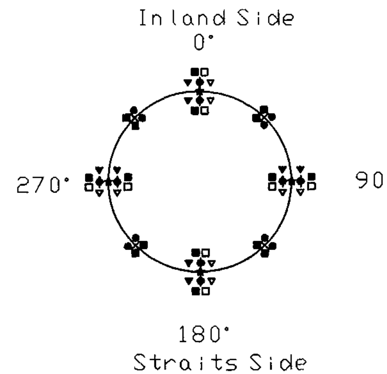

FIG. 8. Placement of Sensors

TABLE 3. Arrangement of Sensors

Purpose (1)	Items to be measured (2)	Number of points (3)
Safety control of underground wall	Stress in concrete in both directions	36
	Stress in rebars in both directions	224
	Lateral pressure	40
	Ground-water pressure	91
	Deformation of wall	4
	Displacement of top of wall	8
Safety control of inside excavation	Ground-water pressure	22
	Displacement of strata	19
	Water level at deepwells	4
Hydration heat	Stress in concrete in inner wall	5
	Hydration heat in inner wall	35

become a large value slightly before the ground might have a failure like heaving. Displacement of the ground below the bottom of the foundation was thus decided to be measured in order to secure the safety of the excavation work as well as to monitor the behavior of the structure under the cable pull. Sensors used here are sliding-type ground-displacement meters. In this instrument, a bore hole is first drilled (usually the hole was reused from the geological survey), and a rod was let down for its tail to be anchored to the bottom of the hole. Sensors were then placed to a designated depth where the sensor was to be fixed to a layer of the ground, and displacement of the ground could be detected as slippage between the sensor and the rod. The number of sensors per rod is usually limited to less than five.

Fig. 11 shows the relationship between the uppermost displacement at the center of the foundation and the depth of excavation; the total number of the holes for sensors is four. It is clear that upward displacement takes place when the excavation comes nearer than 20 m.

Fig. 12 shows the vertical distribution of the displacement, from which the area for significant displacement to occur is concluded to be limited to being near the bottom. These figures include simulated values for two cases. One uses the original modulus of the ground and another uses three times that value. Note that the measured data are actually smaller than these simulated cases. This reasons are probably that the obtained modulus of the ground beforehand is smaller than the actual one, and that transmittance of strain in the ground does not go as far as would be expected by the model.

ROLLER-COMPACTED CONCRETE WORK

This section briefly reviews the application and construction methods involving roller-compacted concrete on the Akashi-Kaikyo Bridge Project.

Application of RCC

After the inside excavation was down to −61 m (volume 330,000 m^3), this huge hole had to be filled with concrete because the foundation needed weight to resist the cable pull. An alternative design would be that the front portion be made with many RC bulkheads and only the rear portion be filled with concrete so as to stabilize the foundation, but this design was

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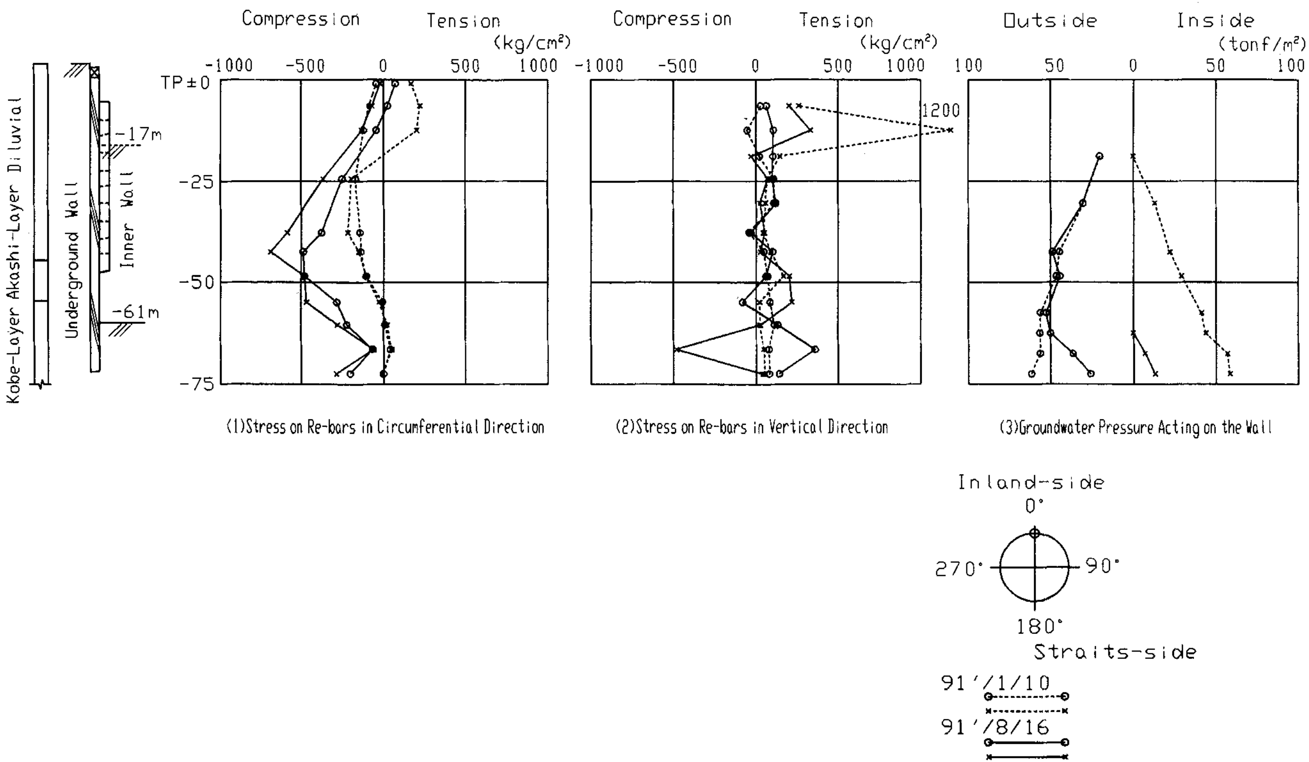

FIG. 9. Stress and Ground-Water Pressure

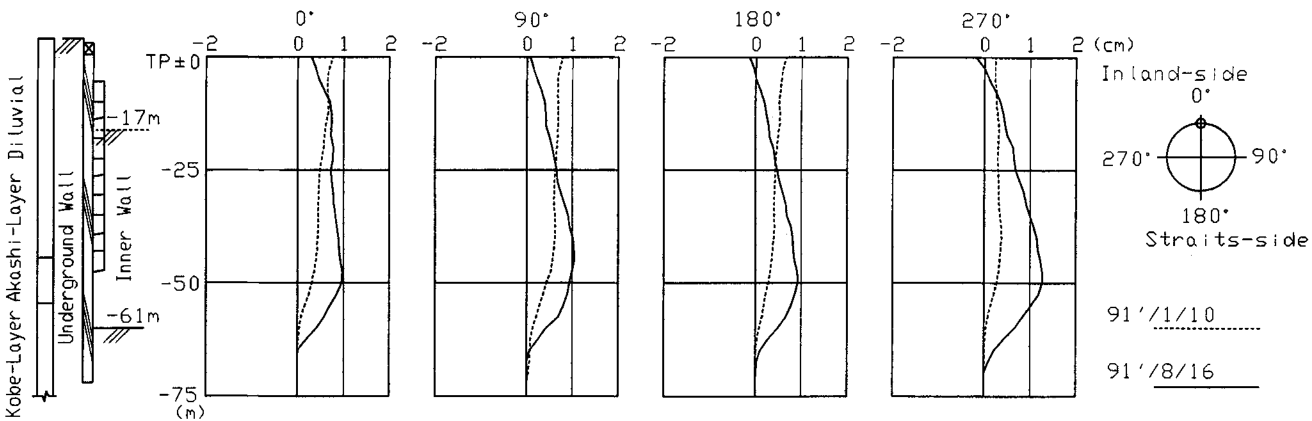

FIG. 10. Deflection of Underwater Wall

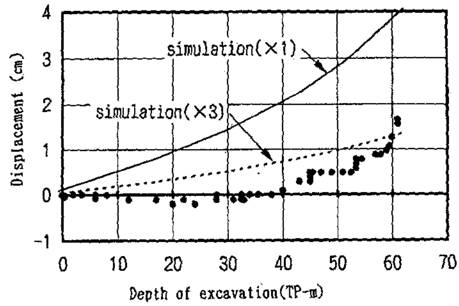

FIG. 11. Displacement versus Depth of Excavation

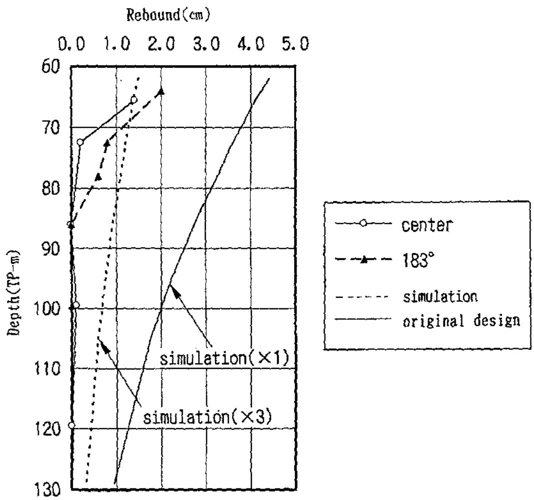

FIG. 12. Vertical Distribution of Displacement

found to be more expensive and to require a longer construction time. Thus, the total-mass-concrete method was chosen. In this, however, the generating stress is so small that no reinforcement is needed in this bulk portion, and it was judged important to reduce the hydration heat of concrete as far as possible. For these reasons, the RCC method was found to be suitable for the volume of about 240,000 m^3.

Execution of RCC

RCC, which is very lean concrete with more coarse aggregate and less cement, was originally developed for dam concrete, and the maximum size of the coarse aggregate used in Japan has been 150 mm. It was, however, difficult for the Akashi-Kaikyo Bridge Project to obtain 150 mm aggregate, since only 40 mm was available. Accordingly, an investigation about mixes and execution methods for RCC with 40 mm aggregate was conducted. The 90-day design strength was 8.5 MPa, with a target mix of 17.6 MPa and a vibrating compaction value of 30 $\pm$ 15 s. The water-to-cement ratio was 0.77, with 130 kg of cement, 100 kg of water, 687 kg of sand, and 1,502 kg of aggregates per cubic meter.

FIG. 13. View of Completed Wall while Placing Mass Concrete in Anchorage 1A by RCC Method

In the actual concreting, the circular shape (in this stage, the inner diameter had been reduced to 76.6 m because of the presence of the retaining-wall system) was divided into two areas, and concrete was to be placed in each area every two days, as shown in Fig. 13. Namely, one area was occupied with the concreting while another area was used for transportation of the concrete. The concrete produced at the site plant was lowered into the foundation with the cranes, which had been used for lifting up the excavated material, and was then carried to the placing area with dump trucks. Here, the concrete was first spread with bulldozers and then compacted with vibrating rollers as if the whole task had been a kind of earth work. Because RCC has no slump, internal vibrators, which are widely used for other types of concreting, do not work.

CONCLUSION

It had often been said in the survey and design states of the Akashi-Kaikyo Bridge Project that construction of 1A foundation was one of the toughest technical challenges in the project, because a deep foundation with large plane area had to penetrate into the bearing layer in spite of the presence of thick soft strata. But its construction was successfully finished in September 1992 after four years of work, including reclamation at the beginning. Some of the important factors for this success are concluded to be quality of the underground wall and inside excavation based on the real-time information about the bedrock and the retaining wall.

When a huge foundation has to be sunk deep through a thick, soft layer, the method adopted at 1A will be one of the best available solutions. The writers hope that this paper can help those who are responsible for similar challenging projects.

Most evident on this project, and on the Honshu-Shikoku Bridge System as a whole, is a systematic evolution and massive scale up of a world technological heritage that extends over a century of suspension-bridge construction. It thus seems appropriate that this paper was first presented at the 1993 ASCE Construction Congress in San Francisco, home of the Golden Gate Bridge, the 1930s long-span suspension bridge pioneer and world record holder for three decades after that.

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