Tokyo Gate Bridge

— Design and Construction of Steel Pipe Sheet Pile Foundation—

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Project Outline

At the Port of Tokyo, construction project has begun on an 8-km section of the Tokyo Port Coastal Highway to enhance the smooth distribution of international cargo between the Port of Tokyo and its coastal cities. Constituting a segment of this section of the highway, the Tokyo Gate Bridge is a large-scale structure composed of a main bridge (continuous 3-span truss-box composite bridge) that spans the No. 3 fairway of the Port of Tokyo and two offshore-onshore approach bridges (continuous multi-span steel slab box girder bridges) at both sides of the main bridge. The bridge has a total length of 2.9 km and is scheduled to be completed in fiscal 2011.

Because the construction area of the Tokyo Gate Bridge is located at a layer of alluvial clay (N value ≈ 0) that is 30 m or more thick, the foundation of the bridge rests on bearing strata of sand and gravel located at the deepest section of the No. 7 stratum. And since the foundation lies at least 65 m below the sea mud line, it is built as a great-depth type structure. Because the foundation is required to possess a seismic deformation capacity suitable for expected seismic motion between 534.7 Gal and –434.2 Gal and because economical structural sections must be built, it was decided that a large-scale steel pipe sheet pile well foundation (pipe pile diameter: 1,500 mm; checkered steel plates + high-strength mortar-filled interlocking joints) is adopted.

In this article, the large-scale steel pipe sheet pile well foundation that was adopted for construction of the Tokyo Gate Bridge is outlined and its construction method is discussed.

Two features should be cited regarding the construction of the Tokyo Gate Bridge. One is that the bridge spans the No. 3 fairway of the Port of Tokyo (crossing width: about 310 m; below-girder clearance: A.P. 54.6 m), and the other is that the construction work had to adapt to impediments posed by the restricted area surrounding Tokyo International Airport (A.P. +98.1 m). In order to satisfy these restrictions at both the design stage and the construction stage and to construct both a safer and higher-quality bridge through the pursuit of structural and economical rationality, it was necessary to adopt the latest in bridge technology: steel slabs utilizing large-size ribs, truss welds attained by eliminating the use of splice plates, BHS (bridge high-performance steel) products, large-size base-isolated shoes etc.

Design of the steel pipe well foundation was made in accordance with the *Specifications for Highway Bridges* (Japan Road Association) and the *Design and Construction of Steel Pipe Piles* (Japanese Association for Steel Pipe Piles). These two specifications state that the required performances of the foundation is constantly satisfied with regard to bearing force and overturning and that, in level 1 seismic motion, the structural materials remain within the allowable stress and the horizontal displacement is within 50 mm, and that, in level 2 seismic motion, the material remains within the yielding value and the yielding area of the foundation ground is within 40%.

Foundation Structures

The substructure of the Tokyo Gate Bridge (total length: 2.9 km) is composed of two abutments and 21 piers (Fig. 1), of which 9 piers are located in the offshore section (extension: about 1.6 km) of the substructure. Of the offshore piers, the main piers (MP2, 3) are of the RC wall type (Photo 1) and the side-span piers (MP1, 4) are of the RC hollow type.

As shown in Fig. 2, a layer of soft alluvial clay (AC2 layer, N value ≈ 0) is thickly deposited over the foundation ground in the vicinity of the steel pipe well foundation. Further, the layers that serve as the bearing strata of the piers are located at great depth: the gravel layer (Dg1 layer) for the CP9~MP2 piers is located at A.P.–75 m or deeper, and the sand layer (Ds2 layer) for the MP3~WP6 piers at A.P.–50 m or deeper. The gravel layer at A.P.–75.5 m mostly serves as the bearing stratum of the CP9~MP2 piers, and the sand layer in the vicinity of A.P.–54.5 m~–50.5 m, shallower than the above, serves as the bearing stratum of the PM3~WP6 piers.

Photo 2 shows the driving of the pipe piles. Each pile is first driven into the bearing stratum using a vibratory hammer and then a hydraulic hammer. The hammers used are IHC-S280 and IHC-S200. The planned embedding length to the bearing stratum is 3.0D~3.2D. The outer diameter of the steel pipe piles used is 1,500 mm, and the wall thickness of the pile tip is 17 mm. Photo 3 shows the construction of the steel pipe sheet pile well foundation.

Fig. 1 Entire Drawing of Tokyo Gate Bridge

Bridge type: 3-span truss-box composite bridge (760 m)
Road classification: Type 4, Class 1
No. of lanes: 4 lanes (for both directions)
Total extension: 2,933 m (offshore section: 1,618 m)
Design speed: 50 km/hr
Planned completion: 2011 (project undertaking in 2001 by the Ministry of Land, Infrastructure, Transport and Tourism)
Total expenditure: ¥141,000 million

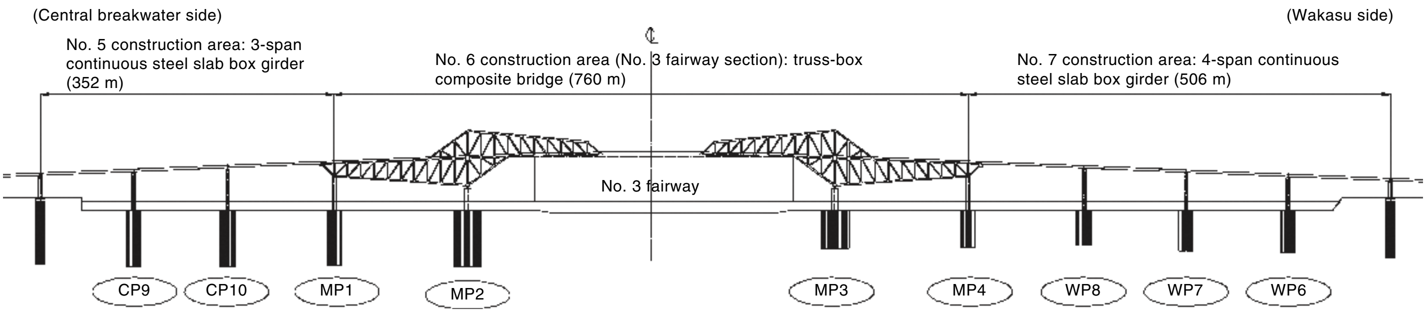
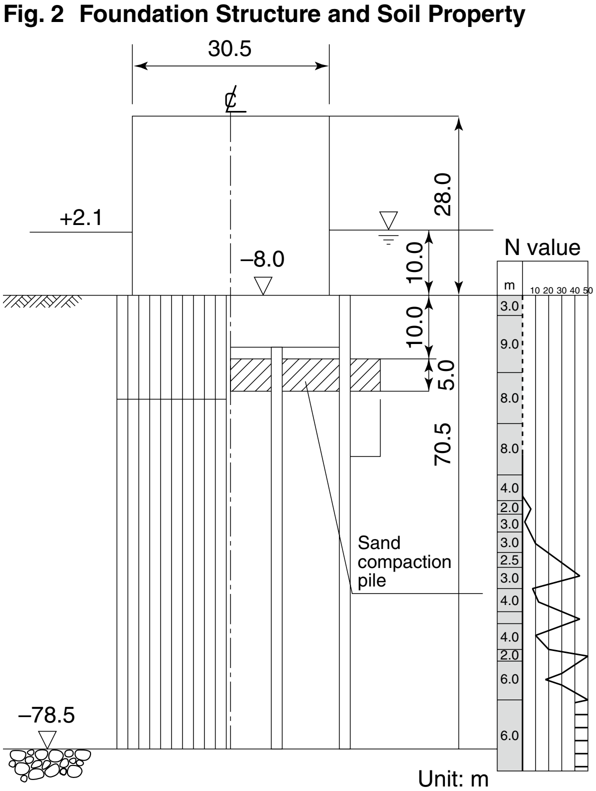



Photo 1 Construction of MP2


Photo 2 Driving of steel pipe piles


Photo 3 Construction of steel pipe sheet pile well foundation

Fig. 2 Foundation Structure and Soil Property



Loading Tests for Vertical Bearing Capacity

In applying steel pipe sheet piles (diameter: 1,500 mm) in the construction of the Tokyo Gate Bridge, on-site loading tests were carried out in 2003 prior to the start of the construction work. The aim of the tests was to clarify the bearing mechanism of large-diameter steel pipe sheet piles that previously had incurred problems in similar work implemented in Tokyo Bay, and to modify conventional design and construction management of the foundation.

Three kinds of loading tests were carried out: dynamic loading tests (DLT), static loading tests (SLT) (horizontal and vertical) and rapid loading tests (STN), which led to the following results:

- Pile tip resistance by means of wave matching analysis from the dynamic test
- Skin friction resistance from the static loading test
- Set-up ratio (ratio of pile-tip static resistance during installation to tip resistance after ground recovery) of tip resistance from the dynamic and static loading tests
- Relation between the skin friction resistance and the N value from the rapid loading test

(Refer to Tables 1 and 2, Fig. 3)

In relation to the design of the steel pipe pile well foundation, the embedding length of the steel pipe piles into the foundation layer and the enclosure ratio of the pile tip are prescribed. Further, the ground reaction coefficient, vertical-direction ground reaction and other factors described in the *Specifications for Highway Bridges* are confirmed. Photo 4 shows the static loading test. Utilizing four pipe piles as the reaction pile, test pipe piles were pressed into the bearing stratum to a depth three times the pile diameter by means of a multi-cycle system employing hydraulic jacks. The hydraulic jack capacity was 48,000 kN for pile ④, and

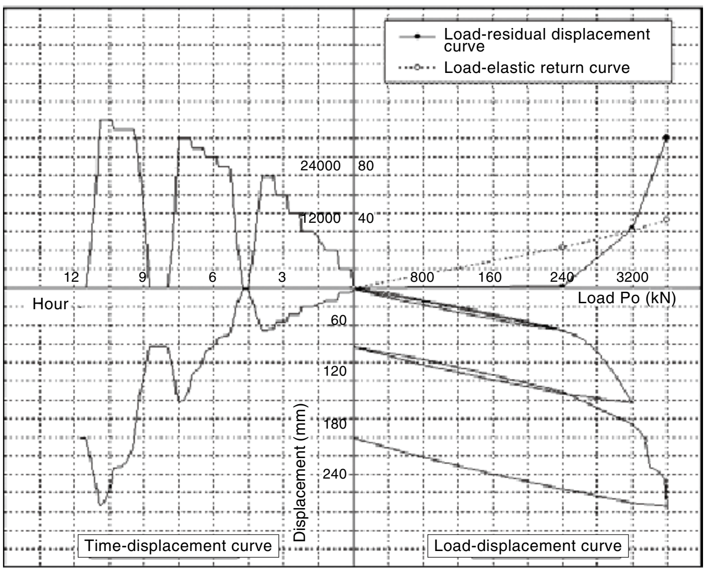
Table 1 Test Piles and Kinds of Loading Tests

Test pile No.	Pile diameter D (mm)	Pile tip depth A.P. (m)	Test kind
①	ϕ1500	−86.0	DLT
②	ϕ1500	−78.5	DLT
③	ϕ1200	−72.6	DLT, STN, CPT
④	ϕ1500	−73.5	DLT, SLT, HLT, CPT
⑤	ϕ1500	−86.0	DLT, SLT, CPT
⑥	ϕ1500	−86.0	DLT
⑦	ϕ1500	−86.0	DLT
⑧	ϕ1800	−89.0	DLT
⑨	ϕ 800	−42.0	—

Table 2 Kinds and Objectives of Loading Tests

Kinds of loading tests	• Static (vertical) loading test • Static (horizontal) loading test • Dynamic loading test • Rapid loading test
Test parameters	• Pile diameter • Embedding length to bearing stratum • Soil property of bearing stratum (sand or gravel)
Examination of design values	• Embedding length • Tip bearing capacity • Skin friction resistance • Ground reaction coefficient (vertical and horizontal directions) • Coefficient in bearing capacity calculation equation • Enclosure ratio and correction coefficient of safety factor

Fig. 4 Results of Static Loading Tests



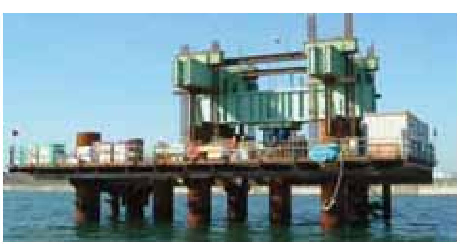
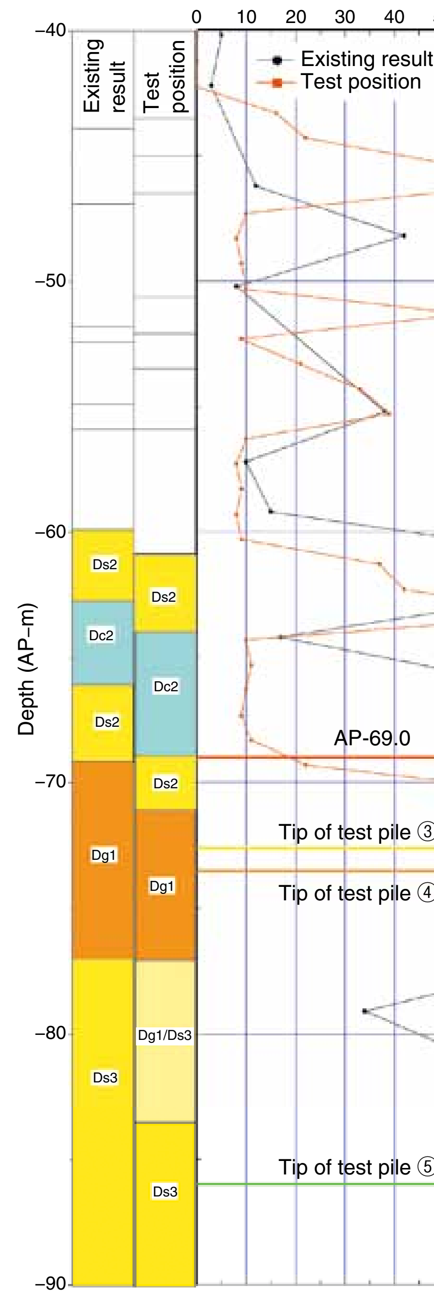


Photo 4 Static loading test

Fig. 3 Relation between Bearing Stratum Thickness and N-value at Test Position



56,000 kN for pile ⑤.

Fig. 4 shows the relation between the applied load and the vertical displacement of pile ⑤. The maximum load was 36,000 kN, and the maximum displacement, 280 mm. Displacements up to about 60 mm were within the elastic range, and the yielding load was 2,000 kN. The strains were measured at 13 sections of the pile, and the axial force distribution and the skin friction resistance were calculated from the measurement results. Fig. 5 shows the distribution of the axial force of the pile. The axial force was calculated from strain meters set in the pipe pile. The difference in axial force between the respective measured sections can be converted as friction resistance. The skin friction resistance is somewhat low in the upper clay layer, but becomes larger in the lower layer. Fig. 6 shows the relation between the pile tip bearing capacity and the total load displacement, which were obtained by separating from the total force. It can be seen from the figure that the skin friction resistance is considerably larger than the tip bearing capacity for pile ⑤. Further, the friction resistance of both the inside and outside surfaces of the pile can be separated from the friction resistance at the pile tip. It was found in the test that the inside-surface friction resistance becomes considerably larger than the outside-surface friction resistance and that the sand and gravel inside the pile are compacted.

As shown in Fig. 7, in the rapid loading test, the 160-ton reaction mass affixed to the pile head was lifted upward at an acceleration of about 20G by the combustion pressure of the propellant, and the load was pseudo-dynamically applied to the pile head using the reaction thus generated by lifting with a loading time of about 0.1 second. This test offers advantages such as shorter test times and the elimination of reaction piles. Photo 5 shows the loading device. The axial force and acceleration distribution are calculated using a load cell, strain meter and accelerometer installed on the pile. Fig. 8

Fig. 5 Vertical Distribution of Axial Force

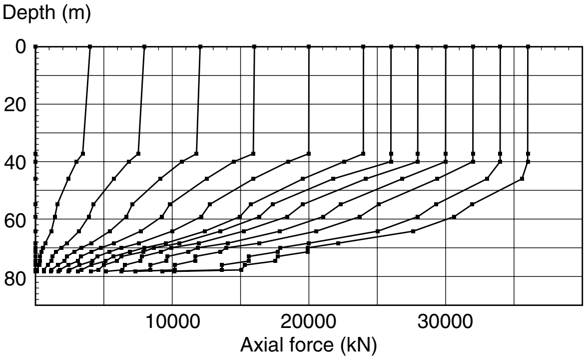


Fig. 6 Tip Bearing Capacity and Total Bearing Capacity

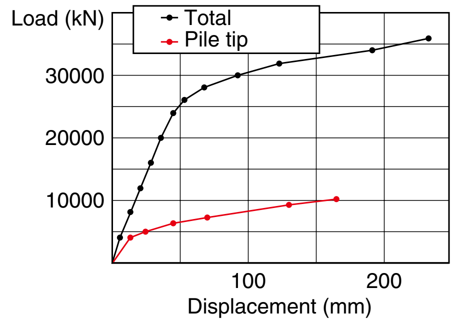


Fig. 7 Structure of Rapid Loading Test Device

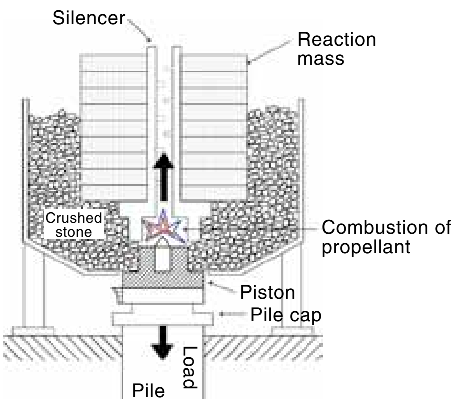


Photo 5 Loading device

shows the relation between the measured load and the displacement. Fig. 9 shows the model used to calculate the pile tip bearing capacity and the skin friction resistance. Calculation is made by setting the pipe pile as the elastic body and linking the peripheral ground, spring and dashpot while at the same time changing the ground constant by using the pile head input load so that the wave forms of both the measurement and the calculation agree each other. Fig. 8 shows the temporal changes of both the load and the displacement calculated together with the measurement results. Further, the input wave (hammer load) and the reflection wave (ground resistance) can be calculated from the axial force and acceleration. These value forms are shown in Fig. 10.

Fig. 11 shows the axial force and skin friction resistance of the piles as obtained from wave-form matching analysis. While the design of the steel pipe sheet pile well foundation was basically implemented according to the method described in the *Specifications for Highway Bridges*, various values applied in the design were modified based on the loading test results. Modification was made mainly of the following:

- The driving depth of pipe piles with an outer diameter of 1,500 mm (D) was set at 3D. The apparent enclosure ratio was 53% for the sand layer and 74% for the gravel layer. When a cross rib was attached to the pile tip, the pile tip resistance increased by about 30%.
- The skin friction resistance was estimated to be added to the pile-inside resistance, and the value shown in Table 3 was adopted.
- It was found in the horizontal loading test that the deformation coefficient of the ground was 2~3 times larger than that described in the *Specifications for Highway Bridges*.

Loading Tests for High-strength Steel Pipe Sheet Pile Joints

Commonly, the deformation strength of a steel pipe sheet pile well is obtained from the bending rigidity of the steel pipe sheet pile structure and the shear resistance of the interlocking joints. In the case of adopting commonly applied pipe-to-pipe joints, the pipe piles are interlocked to each other by the use of steel pipes with a diameter of 165.2 mm, and mortar with a compression strength of about 20 MPa is used to fill the interlocked pipe joints. In the current Tokyo Gate Bridge project, it was necessary to increase the rigidity of the joints so that the horizontal displacement of the steel pipe sheet pile well foundation could remain within its allowable value during earthquakes.

In order to increase the rigidity of the steel pipe sheet piles, high-strength mortar was used to fill the joint openings (Fig. 12) to increase the adhesion between the mortar and the pipe joints. In the current project, the strength of the fill mortar was increased to more than double (40 MPa) the conventional level, and, further, checkered steel plate was used as the material of the pipe joints to increase the adhesion. As a result, it was confirmed that sufficiently large shear resistance could be obtained by the use of these two approaches.

As the results of the model loading test, a maximum shear resistance of about 1,640 kN/m was obtained as shown in Fig. 13. The values obtained in the test were about six times the upper limit (200 kN/m) of the

Fig. 8 Matching Condition for Wave Form

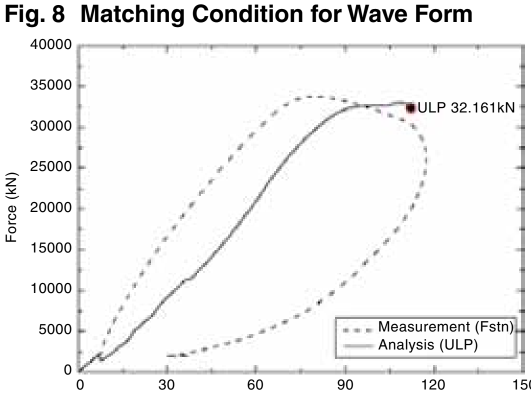


Fig. 9 Calculation Model for Tip Bearing Capacity and Skin Friction Resistance

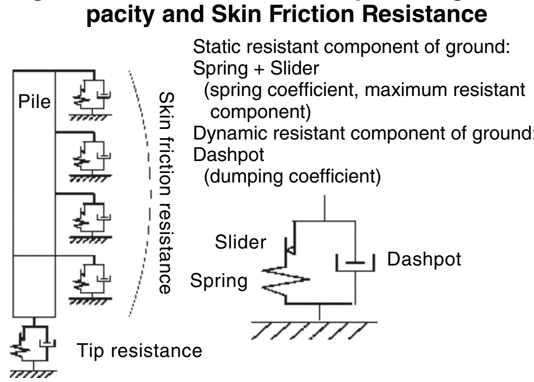


Fig. 10 Input Wave Form Fd and Resistance Wave Form Fu

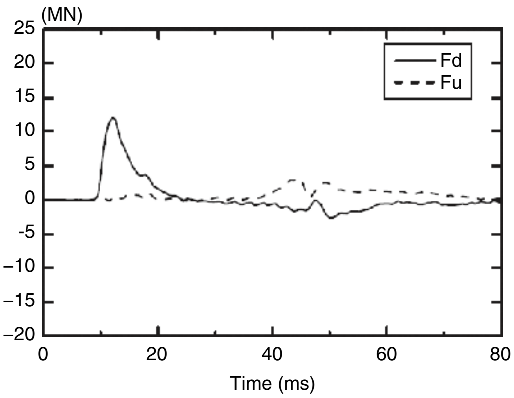


Fig. 11 Results of Wave Form Matching Analysis

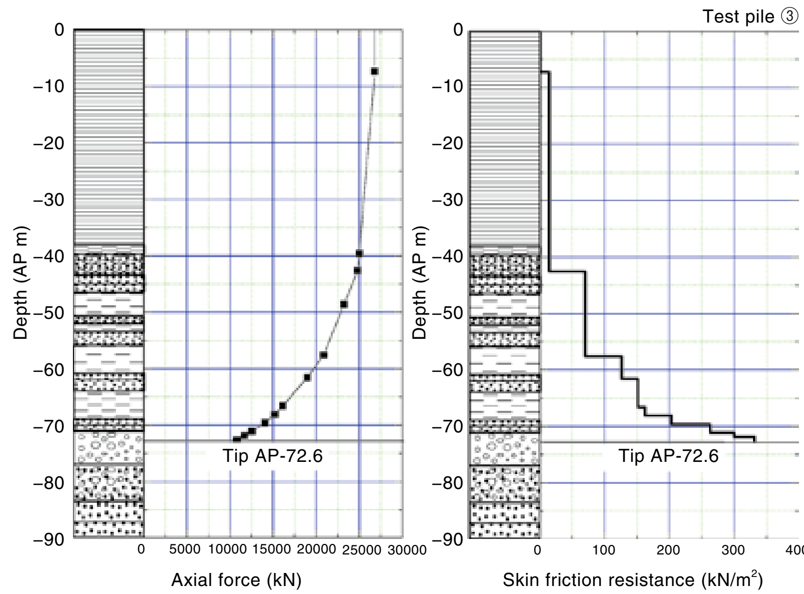


Table 3 Fixing of Skin Friction Resistance

	Fixing by *Specifications for Highway Bridges* (kN/m²)	Fixing by loading test result (kN/m²)
Sandy soil	2N (≤100)	2N (≤100)
Clay soil	10N (≤150)	10N (≤150)
Gravel layer	2N (≤100)	3N (≤150)

Fig. 13 Relation between Load and Displacement

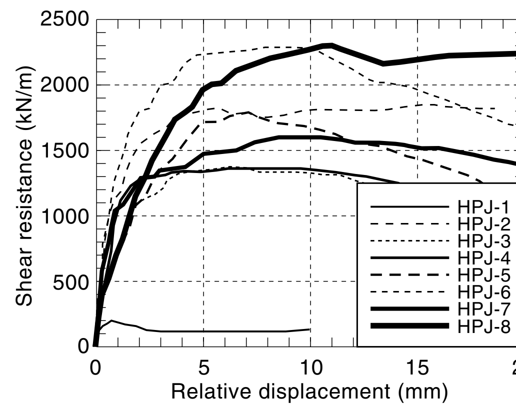
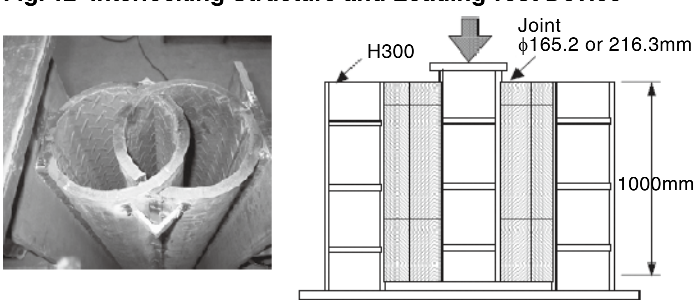


Fig. 12 Interlocking Structure and Loading Test Device



shear resistance described in the common design specifications for steel pipe joint material (*Manual for Design and Construction of Steel Pipe Sheet Pile Foundations*, Japan Road Association).

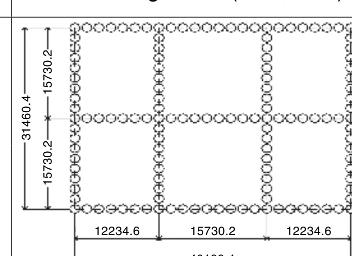
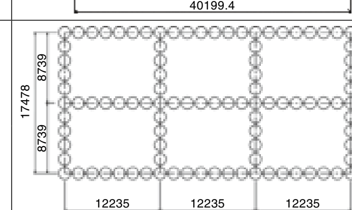
The configuration of the steel pipe sheet pile well foundation designed in the initial stage was greatly reduced through the appropriate use of the loading test results and a reexamination of the ground coefficients, which allowed for economical construction (Table 4).

Reduced Construction Cost and Improved Quality

The steel pipe sheet pile well foundation of the Tokyo Gate Bridge was constructed on soft ground, and, further, large sectional force and deformation capability during earthquakes were required in its construction. Thus in the design stage, the foundation structure necessarily became quite large in terms of structural configuration.

After acquiring knowledge of the bearing mechanism at the construction site from the loading tests conducted on large-diameter steel pipe sheet piles, various design factors were reconsidered. Based on these, a more compact foundation configuration was successfully achieved by the combined use of large-diameter steel pipe piles and interlocking joints made of checkered steel plate filled with high-strength mortar. Similarly, the construction management procedures were also determined by the effective use of the loading test results. As a result of these considerations, the structural dimensions of the steel pipe sheet pile foundation were reduced, thereby simultaneously producing greater reductions in construction cost and improvements in structural quality.

Table 4 Comparison of Designs of Steel Pipe Sheet Pile Well Foundation

	Planar configuration (MP2, MP3)	No. of pile / Pile length
Initial design		136 / 84.5 m
Final design		98 / 81.5 m