Construction of a Floating Swing Bridge—Yumemai Bridge

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1. Introduction

The City of Osaka has been undertaking the "Techno-port Osaka" project to develop in its waterfront area a new metropolitan center with advanced features for the 21st century. This project covers the construction of three reclaimed islands, Maishima, Yumeshima and Sakishima, in the waterfront area (Fig. 1). As a candidate for the host city of the 2008 summer Olympic Games, Osaka has decided to use Maishima as the main venue for the Games. Yumeshima, which is still under reclamation, is planned for residential, commercial and various amenity facilities. Under these circumstances the City of Osaka planned the construction of the Yumemai Bridge, which is expected not only to contribute to accelerated development and improvement of these reclaimed islands, but also to play an important role in transportation access to the waterfront area.

The channel between Yumeshima and Maishima, called "North Waterway," as a subsidiary to the main waterway located to the south of it, provides passage mainly for small craft. These two waterways are the only routes via which ships and boats can access major facilities in the Port of Osaka. If the main waterway becomes unusable due to an accident or for other unforeseeable reason, the North Waterway needs capacity as an international passage through which even large vessels can navigate. To ensure marine passage in emergency, it was decided to construct a movable bridge over this North Waterway. Compared with a tunnel and an ordinary fixed bridge with large clearance under the girder, a movable bridge is far advantageous in terms of costs, construction time, and land use.

The Yumemai Bridge was constructed as a floating swing bridge, the world's first type of movable bridge (Fig. 2). It comprises a floating bridge over the waterway, transitional girder bridges on both ends of the floating bridge,

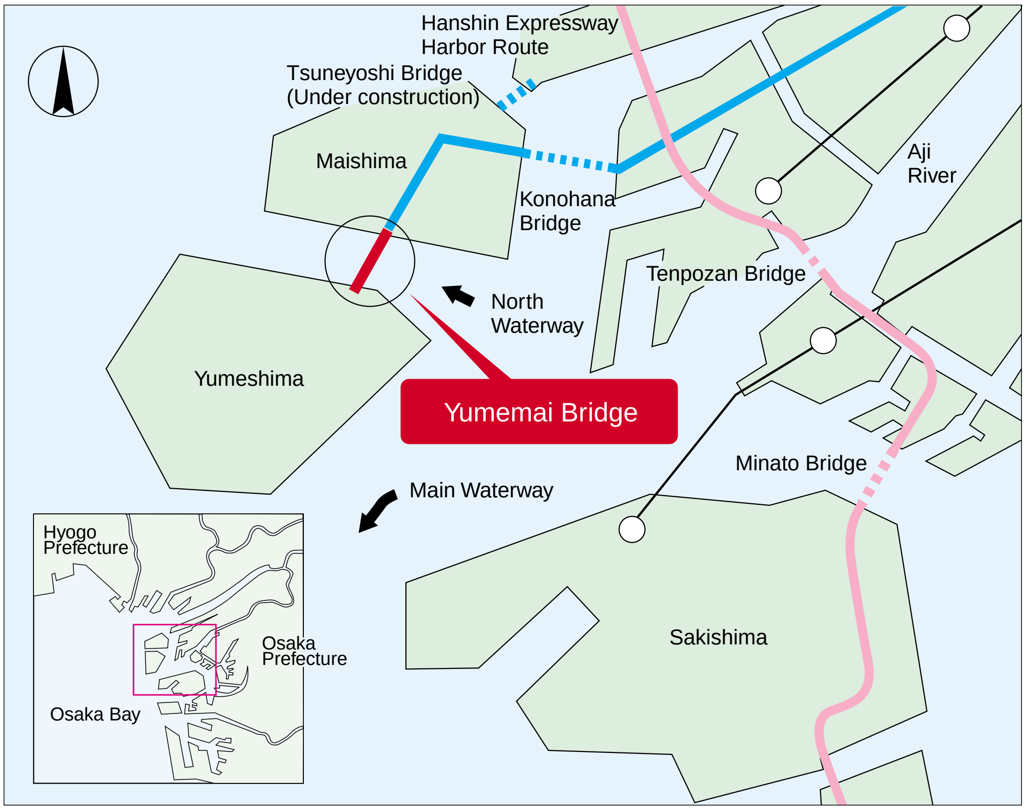


Fig.1 Construction site of Yumemai Bridge

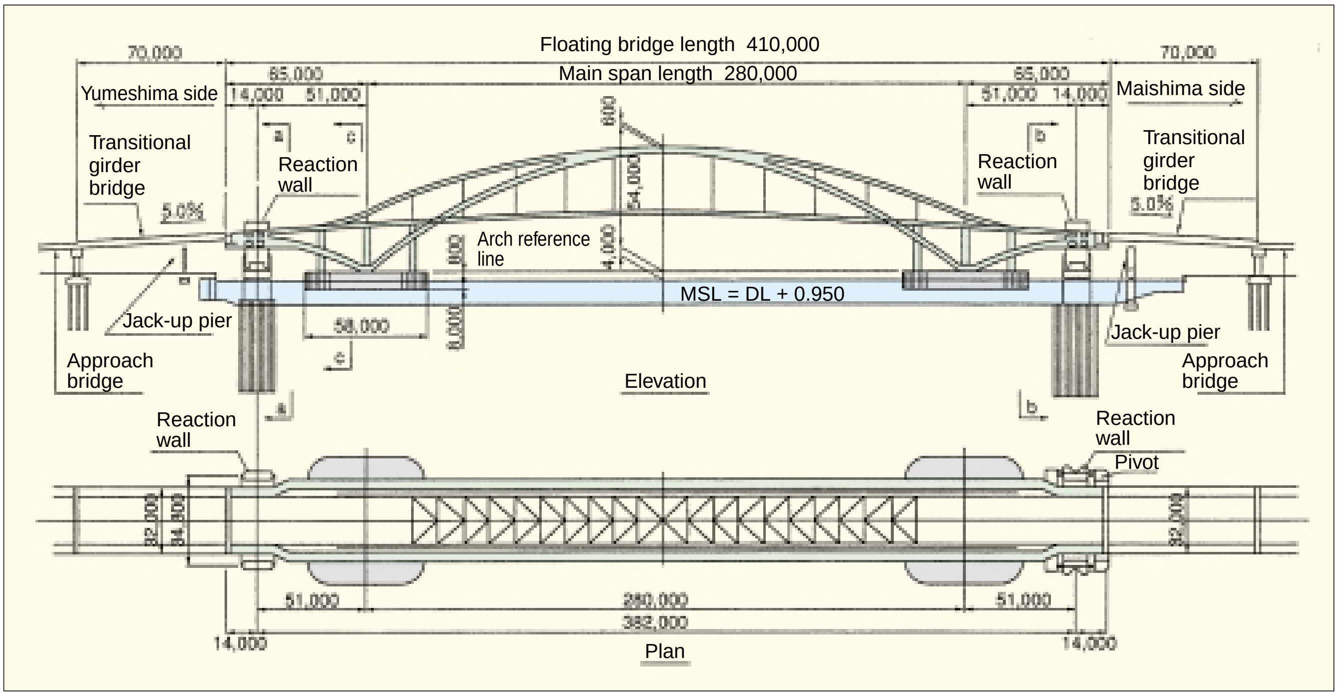


Fig. 2 Superstructure

and approach bridges on the grounds of Yumeshima and Maishima, respectively. This floating bridge, a large arch bridge structure floating on two steel pontoons (58 m ✕ 58 m ✕ 8 m), is horizontally supported by two mooring dolphins with rubber fenders. When positioned for normal service, the floating bridge accommodates a navigation passage width of 135 m (see Photo 1). In emergency, when the main waterway is out of service, the entire floating bridge is swung by tugboats to widen the passage width (200 m or more), enabling the passage of large vessels.[1,2)]

2. Selection of Bridge Type

Normally, the lift bridge, swing bridge, retractable bridge, and bascule bridge are candidates for a movable bridge. These candidates, excluding the bascule bridge, were compared and investigated for suitability to the Yumemai Bridge, which must provide a relatively wide (200 m) navigation passage in emergency. During the preliminary investigation stage, a floating swing type was studied in particular detail from various aspects, including

Photo 1 Panoramic View of the Yumemai Bridge

mooring method. Table 1 summarizes the result of the comparison. A floating swing bridge and a cable-stayed swing bridge were found to have economic advantage over the other types. Closer investigation of these two candidates led us to choose the floating swing type, for the following reasons:

(1) The bridge over the North Waterway will rarely need to be opened. When it becomes necessary to open it, the floating bridge can be swung by tugboats, requiring very little power and minimum drive equipment. In addition, opening by towing is accurate.

(2) Yumeshima is still in the process of reclamation, and ground displacement and subsidence by consolidation are inevitable. With a floating bridge, the influence of ground displacement and subsidence on the bridge and bridge driving system can be minimized.

(3) The bridge is erected at a large dockyard and towed to the installation site. Since the superstructure and substructure can be erected simultaneously, a floating bridge can substantially save on construction time.

For the superstructure of the floating bridge, single-rib arch, double-rib arch, and truss designs were proposed. As the result of comparison, the double-rib arch design was selected because of its superior overall rigidity, and the potential for minimizing wave influence and local distortion by uniformizing the flexural and torsional rigidity along the bridge axis. The aesthetic effect was also taken into account.[3)]

3. Technical Challenges

Various design standards, including the "Highway Bridge Specifications" are conventionally used in bridge design in Japan. In designing this floating bridge, however, these standards alone were not sufficient. It was necessary to establish new design techniques and a new concept of design safety factors. Floating structures have been the focus of investigation in various occasions, e.g., at the time of planning the Kansai International Airport, and in the "Mega-Float Project" led by the Ministry of Transport. Among large floating structures now in service in Japan are oil reservoir terminals at Kamigoto (Nagasaki Pref.) and Shiroshima (Kita-kyushu City). With reference to the data and experience thus accumulated, various technical challenges had to be dealt with in the process of designing the Yumemai Bridge. Fig. 3 is the flow chart showing this process. The major technical challenges[4,5,6)] are listed below:

(1) Since a floating bridge is more vulnerable to meteorological and oceanographic conditions than a conventional fixed bridge, proper environmental conditions, suitable to the characteristics of the installation site, must be set in designing the bridge.

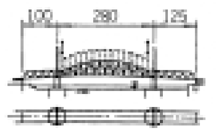
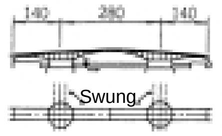
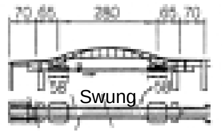
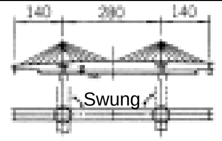
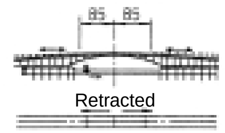
(2) The motion of a floating bridge in winds and waves must be studied in detail, so that the result can be incorporated in the design.

(3) The driving safety and riding comfort of vehicles on the bridge must be maintained against bridge deck geometric line form change caused by tidal flux and bridge motion.

(4) The characteristics of rubber fenders used as mooring shock absorbers must be identified and taken into consideration in the design.

(5) The effect of ground displacement on the bridge structure must be evaluated and taken into consideration in the design.

Table 1 Comparison of Various Movable Bridges

	Lift bridge (Curved chord truss through bridge)	Floating bridge (Pool-type upper side supported bridge)	Floating bridge (Swing type bridge)	Swing bridge (Cable-stayed bridge)	Retractable bridge (Abacus bridge with lifts)
Schematic (Unit: m)					
Economy	×	×	○	○	×
Structural rigidity	○	△	△	○	×
Construction ease	△	△	○	△	△
Movable performance	○	△	△	△	△
Ground displacement	×	○	○	△	△
Serviceability	○	△	△	○	×
Maintenance ease	△	△	○	△	△
Overall rating	△	×	◎	○	×

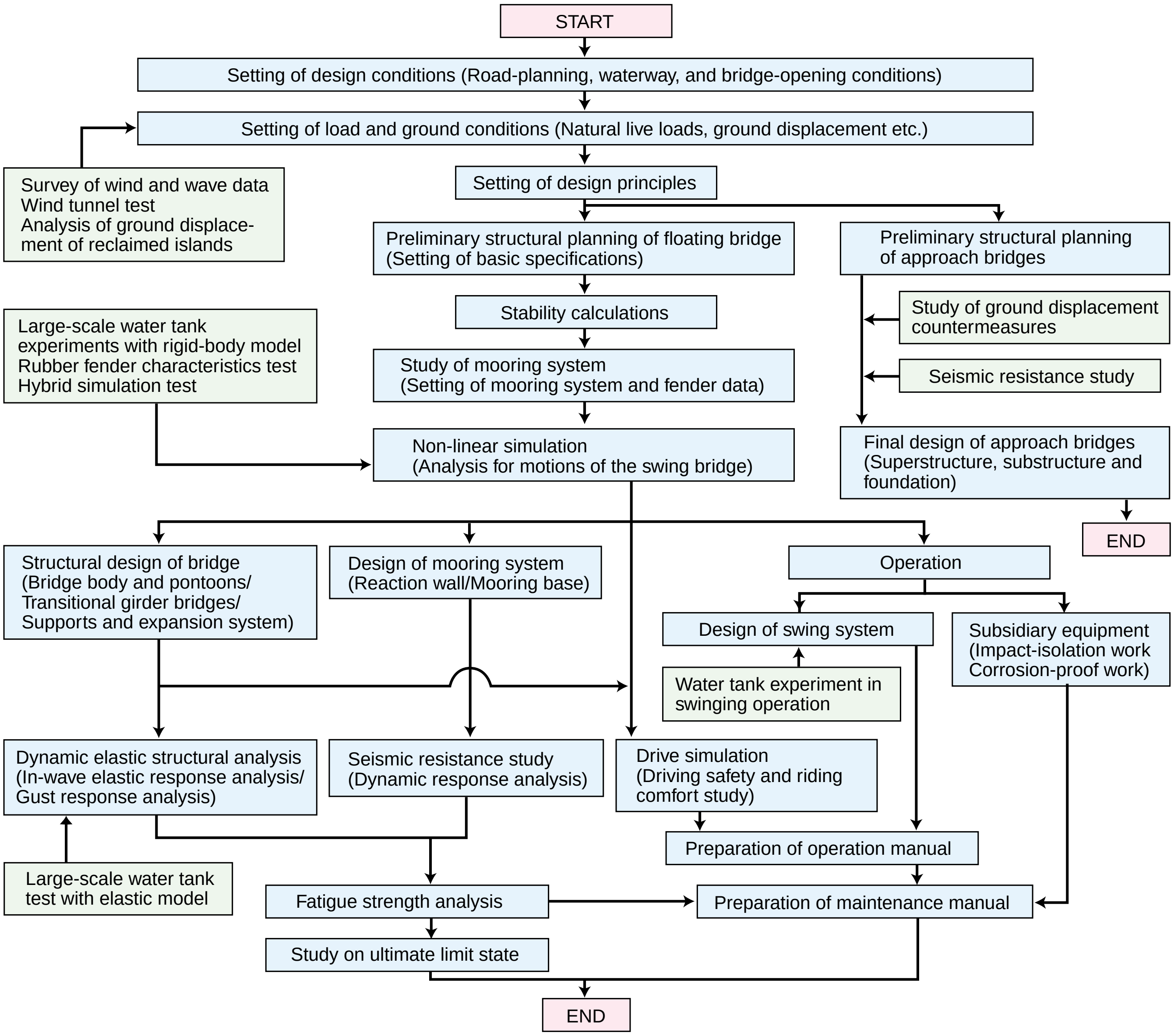


Fig. 3 Flow Chart for Design of Yumemai Bridge

(6) Floating bridges are generally said isolated bridge type. This general notion must be validated to confirm seismic safety; seismic displacement must also be determined and incorporated in the design.
(7) Since this bridge is of the swing type, manuals must be prepared for opening/closing and for maintenance.

4. Design Conditions

4.1 Meteorological and Oceanographic Conditions

The basic design wind velocity (V_{10}: wind velocity at height of 10 m) was set at 42 m/s with a 100-year return period, based on wind velocity data obtained near the installation site and the observation record (1931 to 1995) supplied by the Osaka District Meteorological Observatory. For bridges near the installation site, the regulation sets the traffic safety wind velocity limit at V_{10} = 20 m/s. This value was adopted as the design condition. The wind velocity limit for safe bridge opening/closing was set at V_{10} = 15 m/s, on the basis of the marine operation standard for the Port of Osaka.

As for design tide level, a tidal fluctuation between DL + 4.8 m (design high tide) and DL - 0.52 (ultra-low tide) was assumed, the design datum level (construction datum level) being CDL + 0 m. The design wave was set at H1/3 = 1.4 m, based on typhoon and gale data for the past 40 years (1956 to 1995), the result of wave diffraction calculations for the waterway, and the result of large-scale water tank experiments. Based on wave spectrum observation at the wave observation tower in Osaka Bay, simulation of bridge drift in winds and waves employed the Bretschneider-Mitsuyasu type wave spectrum.

The design tidal current velocity under ordinary conditions was set at 0.2 m/s on the basis of existing data; that under storm condition, for which there was no data, was set at 0.5 m/s by estimation taking bridging site topography into account. As for tsunami, the design tidal fluctuation at the site was set at 2.62 m, and the flow velocity (including tidal current) at 2.6 m/s, on the basis of values set for the regional disaster prevention program of Osaka City.

4.2 Earthquake

"Expected earthquake," taking into account the influences of the active faults, topography, geology and ground condition of the bridging site, was used to determine the bridge's seismic requirements. Specifically, the design considered the two types of expected earthquake waves: one based on the Tohnankaido-Nankaido Seismic Fault Line model, which corresponds to a level II type I earthquake (Plate boundary earthquake) as provided in the "Highway Bridge Specifications," and the other based on the Uemachi Active Fault Line model, which corresponds to a type II earthquake (Inland earthquake).

4.3 Combined Loads, Allowable Stress Increment Factors, and Safety Factors

The design of the Yumemai Bridge is essentially based on existing design standards. However, for combined loads, which are not covered by any of existing design standards, allowable stress increment factors were set using the safety evaluation techniques proposed by the Japan Society of Civil Engineers. Table 2 lists the allowable stress increment factor for each load combination on the floating structure and the mooring dolphins. For each major component, the margin to ultimate collapse was determined and incorporated in the safety factor.

Table 2 Combined Loads and Allowable Stress Increment Factors

(a) Floating structure (Superstructure, pontoon and pivot pin)

Loading status	Combination of loads	Allowable stress increment factor
Standard	D + U + L + I	1.00
Temperature	D + U + L + I + T	1.15
Storm	D + U + W + WP	1.20
Earthquake	D + U + EQ	1.50
Swinging	D + U + W + WP + DR	1.25
Construction and towing	D + U + W + WP + ER	1.25

(b) Mooring dolphin (including reaction wall, reaction wall supporting beam, reaction wall anchor frame, RC dolphin, steel pipe sheet pile well foundation, and floating structure fender installing section)

Loading status	Combination of loads	Allowable stress increment factor
Standard	D + U + GD	1.00
Temperature	D + U + T + GD	1.15
Storm	D + U + W + WP + GD	1.50
Earthquake	D + U + EQ + α • GD (α: coefficient)	1.50
Swinging	D + U + W + WP + DR + GD	1.25

D: Dead load
L: Live load
I: Impact
U: Uplift of buoyancy
E: Earth pressure
W: Wind load
T: Effect of temperature fluctuation
EQ: Effect of earthquake
WP: Wave pressure
PD: Tidal force
GD: Effect of ground displacement
SD: Effect of supporting point displacement
DR: Bridge driving load
ER: Load during erection
CO: Ship collision load
TU: Effect of tsunami

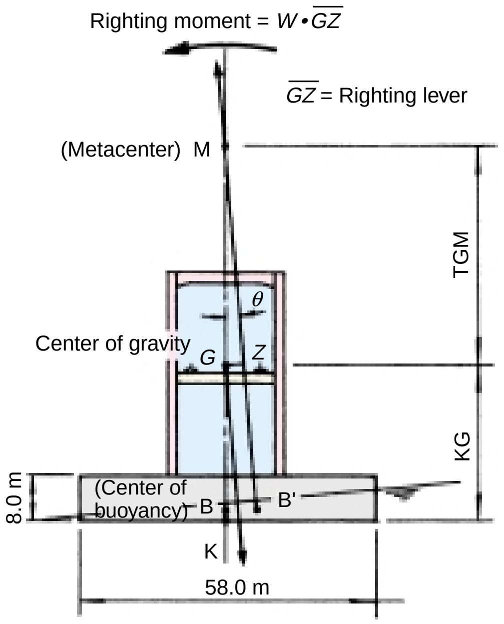


Fig. 4 Centers of Buoyancy and Gravity, and Metacenter

5. Study for Stability

For the Yumemai Bridge, a long-span floating bridge supported on two pontoons, floating stability is a very important concern. Since this bridge must secure a large navigation clearance of 26 m under the bridge girders, the bridge's center of gravity and the wind loading point are necessarily positioned high. To ensure its stability, therefore required careful attention.

For the initial righting moment to secure a satisfactory static stability of the floating structure, it is essential that the vertical distance between the center of gravity and the transverse metacenter (i.e., TGM), shown in Fig. 4, are positive. The greater the TGM value, the greater the stability of the floating structure. Using the basic design dimensions of pontoons, static stability calculations were carried out for three cases: without live load (S1), with biased live load (S2), and with full live load (S3). Table 3 shows the results. For all three cases, the calculated values are larger than those for conventional marine structures and ships, verifying that this floating bridge is extremely stable.

Dynamic stability was evaluated using the following formula:

Area $(A + B) \geq 1.4 \times$ Area $(B + C)$

in which areas A, B and C are as schematically shown in Fig. 5. To satisfy this formula, the righting moment must be at least 1.4 times the inclining moment. The required righting moment inclining moment ratio for securing satisfactory dynamic stability, determined using the pontoon dimensions as parameters, was incorporated in the basic design. Table 3 also shows the ratio for a 58 m $\times$ 58 m $\times$ 8 m pontoon, for each of the above-mentioned three cases.

6. Mooring Method

The floating bridge is supported vertically by the buoyancy of seawater. It must also be supported horizontally to resist such lateral forces as wind, wave and earthquake. Horizontal support is achieved by mooring. The following three different mooring methods were compared and assessed for applicability to this floating bridge:

- Anchor chain mooring
- Submersible mooring
- Rubber fender mooring

Comparison revealed that the rubber fender mooring most effectively restricted the bridge motion, yet was the most economical. Therefore, the focus of our attention was on the two rubber fender mooring methods: reaction wall and link damper. Fig. 6 schematically shows the two mooring methods, and Table 4 compares their characteristics. The wall reaction method has been adopted, due to its superior bridge motion prevention characteristic, and con-

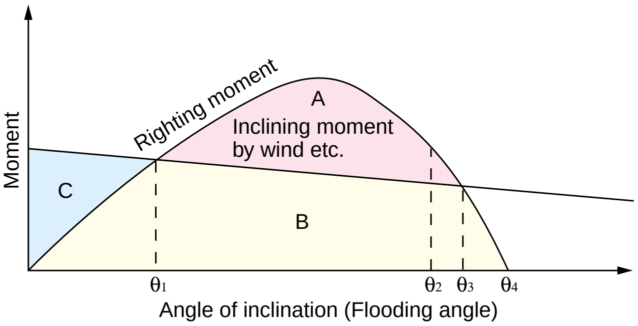
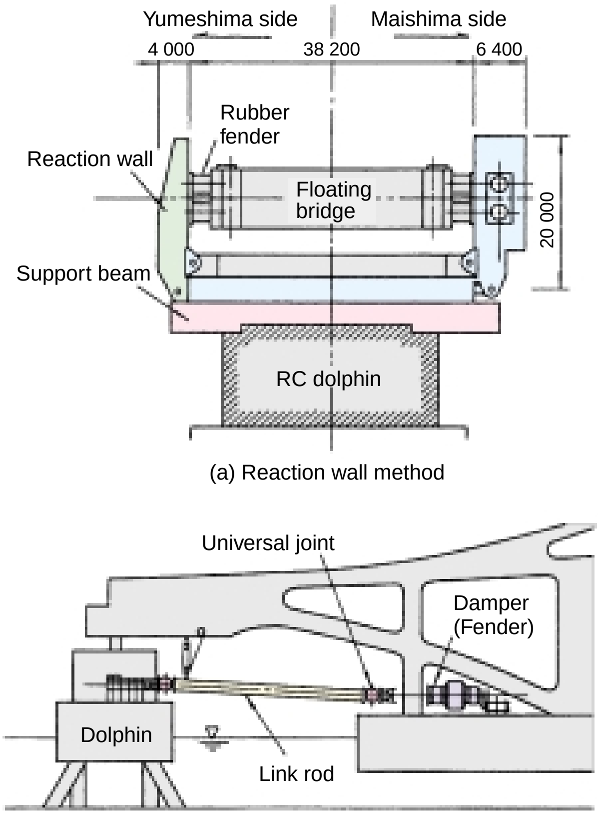


Fig. 5 Dynamic Stability Study



(a) Reaction wall method

(b) Link damper method

Fig. 6 Schematic of Mooring Systems

Table 3 Static and Dynamic Stability Calculation Results

	Case S1 & D1	Case S2 & D2	Case S3 & D3	Condition
Mean draft d (m)	4.80	5.08	5.30	
Displacement Δ (t)	31 445	32 287	34 700	
Height of center of gravity KG (m)	26.38	26.58	26.72	
Transverse metacenter TGM (m)	30.09	27.03	24.92	TGM > 0
Lateral inclination θ (deg)	0.00	1.14	0.25	
DSR = $(A + B)/(B + C)$	1.44	4.97	4.50	DSR ≥ 1.4

(Specific gravity of seawater: 1.025)

Table 4 Comparison of Mooring Methods

Item \ Type	Reaction wall method	Link damper method
Motion of bridge body	Motion in wind and waves is relatively small, since mooring point is at the same level as the center of gravity.	Motion in wind and waves is relatively large, since mooring point is at the same level as the pontoon.
Opening/ closing operation	Release from the mooring system and positioning of the bridge are relatively easy, since they involve only moving the reaction walls.	Rod connection/disconnection requires labor and involves operation of bridge position retaining mechanism.
Technical problem	Steel 20 m high movable reaction wall. Reaction wall operating mechanism, and fixing pin insertion/removal mechanism	Link mechanism Rod connection/disconnection mechanism
Dolphin (Foundation)	Load-acting point is high, resulting in a large moment.	Load-acting point is low, resulting in a small moment.

venience in bridge swinging operation. The constant-reaction rubber fenders used for mooring this bridge have the reaction characteristics shown in Fig. 7.

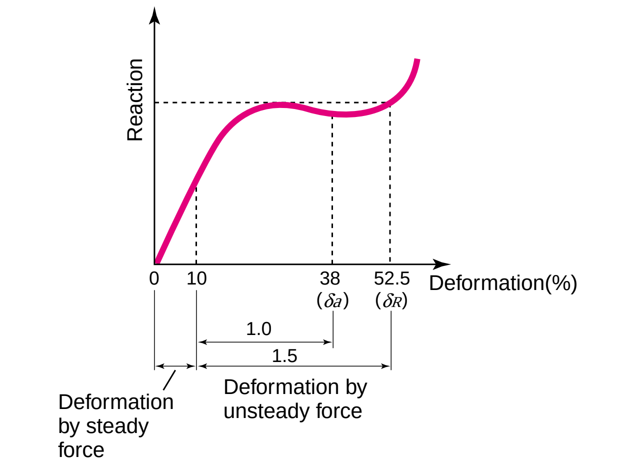


Fig. 7 Relation between Reaction Force and Deformation of Rubber Fender

7. Wind Tunnel Test

In determining the design of each component of a floating bridge, storm wind and wave loads are more influential factors than in the case of a fixed bridge. Therefore, proper evaluation of wind load is necessary. If wind load can be reduced by a relatively simple measure, cost can be saved. In view of this, the static wind load characteristics (mainly drag coefficient) of this floating bridge were investigated by wind tunnel test[7] using a rigid 3D model, and effective wind load reduction measures were sought. (See Photo 2.)

The test revealed that the following measures are effective in reducing wind load:

(1) A corner cut is formed in each side face of the upper and lower arch ribs, so that the aspect angle becomes approximately 30 degrees.

(2) A fairing is provided on both ends of the stiffening girder, and the bottom face of the girder is closed to streamline the girder vertical section.

These measures reduce the drag force by about 20%. The measures shown in Fig. 8 were therefore applied to the basic section of this bridge. For the stiffening girder, a box girder was adopted in lieu of facing plates.

8. Large-scale Water Tank Experiments & Non-linear Computer Simulation

In designing a floating bridge, it is essential to clarify the bridge's drift characteristics in wind and waves, and to obtain accurate drift characteristic values. Since the Yumemai Bridge would be moored by rubber fenders, a

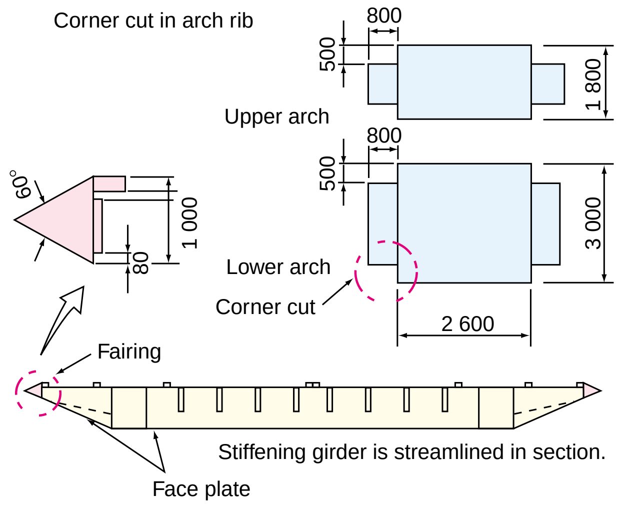


Fig. 8 Wind Load Reduction Measures

new analytical technique had to be developed that takes into account the non-linear characteristic of the rubber fenders. It was also necessary to determine the effect of the relatively flexible bridge structure's elasticity and to investigate opening/closing safety of the mechanical system.

For designing the floating bridge, a structural analysis program was developed, and large-scale water tank experiments were conducted to verify the appropriateness of simulation-based calculations. Hybrid simulation testing was also carried out to clarify the actual behavior of constant reaction rubber fenders with high nonlinearity. Three different large-scale water tank experiments were conducted, and drift simulation results were validated using three different programs which contain the same basic formulas but improved for consistency with respective experiments. Table 5 outlines the large-scale water tank experiments and the hybrid simulation test.

Table 5 Large-scale Water Tank Experiments and Hybrid Test

	Model	Scale	Water tank size (Length × Width × Depth)	Site of experiment
Experiment I	Topographic model, Rigid model	1/80	50m×40m×*	Tsuchiura, Ibaraki pref.
Experiment II	Elastic model	1/40	190m×30m×*	Nagasaki, Nagasaki pref.
Experiment III	Rigid model, Bearing model	1/80	100m×5m×*	Akishima, Tokyo
Experiment IV	Rubber fender model	1/12.5	—	Totsuka, Yokohama, Kanagawa pref.

*: Depth conforms to water depth at site.
For experiment II, height was varied.

Photo 2 Wind Tunnel Test

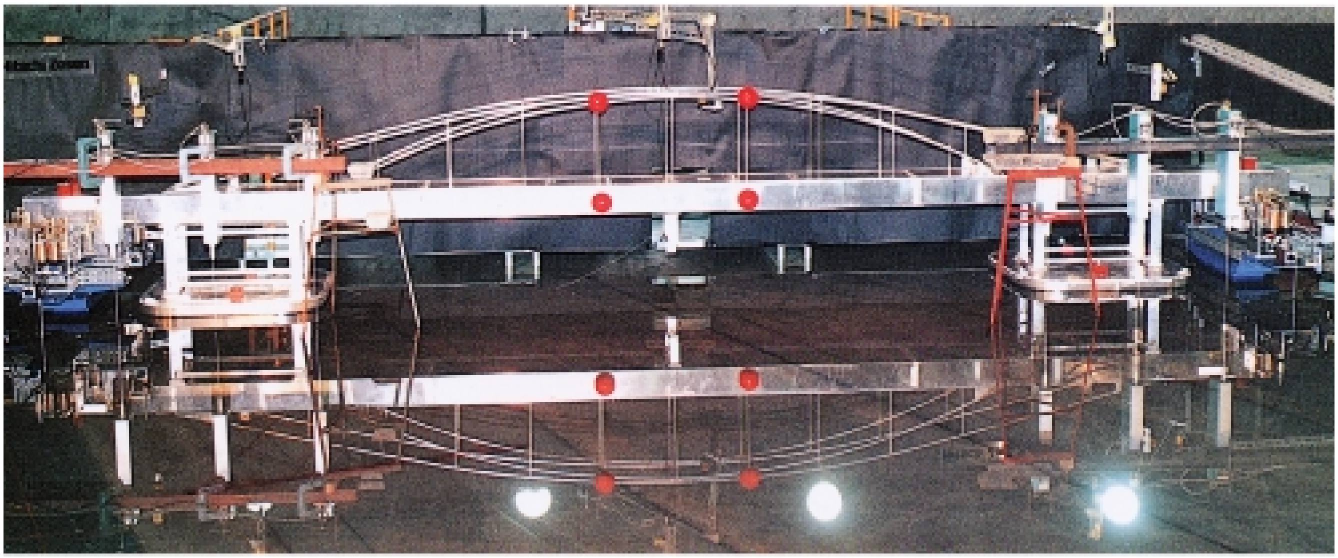


Photo 3 Rigid Model Experiment on In-wave Motion of Moored Floating Structure

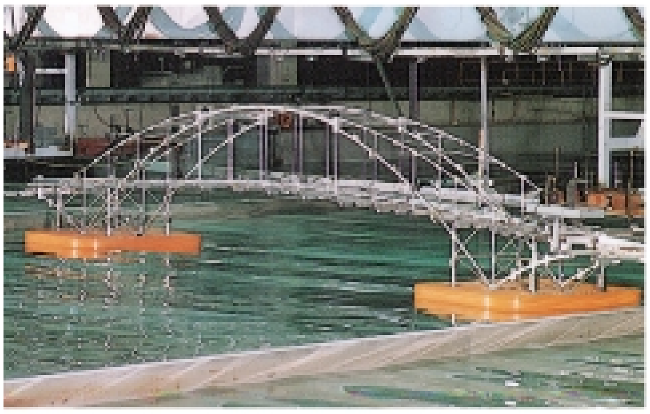


Photo 4 Elastic Model Experiment on In-wave Elastic Response

Photo 5 Experiment in Swinging and Temporary Mooring Operations

- Experiment I[8] (Rigid model experiment in a topographic model) (Photo 3)

  The objectives of the experiment were:

  (1) To clarify the oceanographic conditions at the bridging site, taking into account the influence of wave diffraction and interference, by accurately reproducing the waterway between Yumeshima and Maishima and the seawall structures

  (2) To obtain data regarding the motion of the entire floating bridge, deformation of rubber fenders, etc., and develop simulation techniques and data that could represent such motion and deformation

- Experiment II[9] (In-wave elastic model experiment) (Photo 4)

  Objectives of the experiment were:

  (1) To experimentally investigate the elastic response of the floating bridge in waves

  (2) To experimentally verify in-wave elastic response simulation's applicability to the structural design

- Experiment III[10] (Swinging operation experiment) (Photo 5)

  The objectives of the experiment were:

  (1) To confirm bridge swinging performance, and check loads imposed on the swing mechanism during swinging operation

  (2) To obtain data on swinging operation, such as the tug thrust of a tugboat

  (3) To confirm the temporary mooring force

  (4) To validate the analytical program by comparing the analytical results with numerical analysis results

- Experiment IV[11] (Hybrid experiment)

This floating bridge is horizontally supported by mooring with rubber fenders. Various experiments and simulation analyses were carried out to clarify the motion of the floating bridge in wind and waves. It is known that the reaction characteristic of rubber changes when the rubber is subjected simultaneously to different deformations other than compression. The hysteresis of deformation also changes with loading repetition. Therefore, it was necessary to study how rubber fender characteristics change with deformation, and to verify the appropriateness of the formula for simulation calculations. To these ends, hybrid experimentation was carried out using a scale model of the rubber fender.

9. Driving Comfort Simulation

The vertical load of this floating bridge is supported by the buoyancy of seawater, and the horizontal load by the mooring system. Therefore, in addition to deflection, which is a general problem with ordinary fixed bridges, each of the following changes had to be studied from the viewpoint of vehicle driving safety and serviceability:

(1) Change in longitudinal gradient of the transitional girder bridge decks due to tidal change

(2) Change in longitudinal and transverse gradients of the floating bridge deck due to wind and waves

(3) Change in draft of each pontoon due to live loading

It was necessary to confirm that these changes would cause no problem in regard to driving safety and riding comfort.

At present, there is no regulation or standard specifying the requirements regarding the riding comfort of vehicles on bridges. Therefore, driving on this floating bridge was simulated, and a questionnaire survey was conducted as to the vibration feeling and riding comfort on existing bridges in Osaka. The relations between the simulation and the questionnaire survey results were used as data for relative evaluation of floating bridge riding comfort.[12]

To evaluate driving safety, vehicular lateral and vertical accelerations were calculated by simulation. The result showed that these accelerations would cause no problem in driving safety, considering the long oscillation period of the bridge.

A large bus carrying 36 passengers was run at a speed of 30 to 60 km/h on existing long-span bridges, viaducts in the urban area and ordinary roads in Osaka. Vibration acceleration was measured in the bus, and the 36 passengers were asked to fill in a questionnaire on riding comfort, to obtain the correlation between vibration acceleration and riding comfort. Riding comfort was rated in five grades as

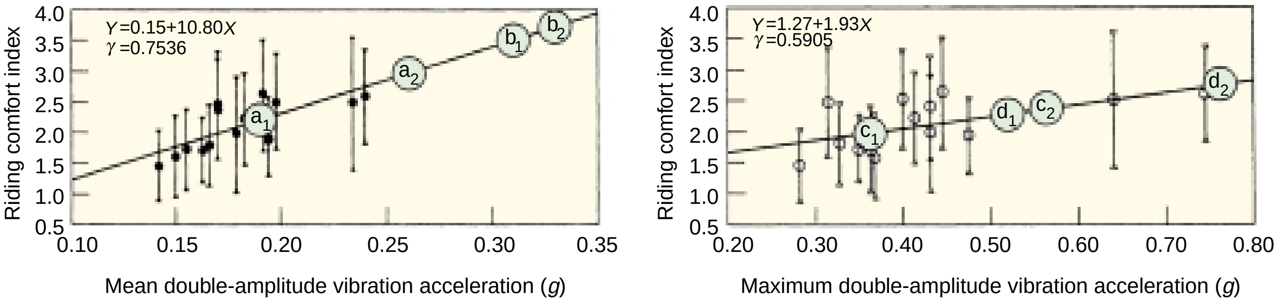


Fig. 9 Correlation between Vibration Acceleration and Riding Comfort Index

shown in Table 6.

As an example evaluation result, Fig. 9 shows the correlation between riding comfort index and mean or maximum vertical vibration acceleration as measured on the bus floor over the front wheels. The correlation shows that only in the worst case passengers may feel peculiar but not annoying vibration. It is reasonable to conclude, therefore, that vehicles can run safely on the floating bridge.

Table 6 Riding Comfort Rating

1: No peculiar vibration is felt.
2: Some vibration is felt which causes no problem.
3: Obviously peculiar vibration is felt.
4: Vibration is considerably large and uncomfortable.
5: Vibration is extremely large, uncomfortable and uneasy.

10. Superstructure Design

In designing the superstructure of this bridge, the sectional force was calculated based on the results of static and dynamic analyses. Fig. 10 shows the models used for static analysis. The buoyancy working on each pontoon was evaluated at the vertical spring set at each node of the pontoon. A floating bridge is subjected simultaneously to wind and wave loads. To study the influence by the elastic response of the floating structure in wind and waves, dynamic gust response in wird and in-wave elastic response analyses were carried out to obtain the sectional force.

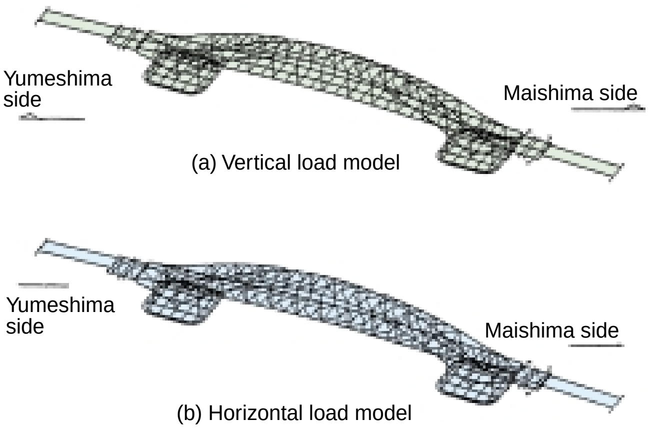


Fig. 10 Static Analysis Models

The sectional forces due to dead load, drift force of waves, tidal force, and lateral inclination caused by winds and waves, thus obtained based on the static and dynamic analyses, were evaluated superposed over each other, to design the superstructure.

The intersection between each support and inside arch forms a corner and generates complex stress. The design section was therefore studied by 3D FEM analysis, to confirm structural safety.

Each arch rib has been designed as a beam-pillar member that receives axial force and in-plane and out-of-plane bending moments. The ultimate load resistance of the design arch rib was estimated in order to confirm that the arch rib collapse load is sufficiently high compared with the load working on the rib, leaving a sufficient safety margin.

11. Design of Pontoons

Considering the shallow waterway at the bridging site and the long span of the floating bridge, pontoons of PC structure would become huge in size, making it difficult for the bridge to allow safe passage of boats. In addition, large thin-wall PC structures are difficult to construct. In view of these aspects, steel pontoons were adopted for this floating bridge.

Fig. 11 shows the pontoon internal structure. The outermost frame of the pontoon is of double-hull structure comprising outer wall and water-tight inner wall, as a failsafe measure against possible water leakage in the event of damage to the outer wall. The water-tight inner wall is installed 3 m inside the outer wall. For safety in case of ship collision, the outermost construction limit was set at 6 m inside from the outer wall. Superstructure supports are positioned within this limit. The stress generated at the base of each pontoon support is too complex to be determined by skeleton analysis only. Stress flow was therefore clarified by analyzing the FEM model of the entire pontoon shown in Fig. 12.

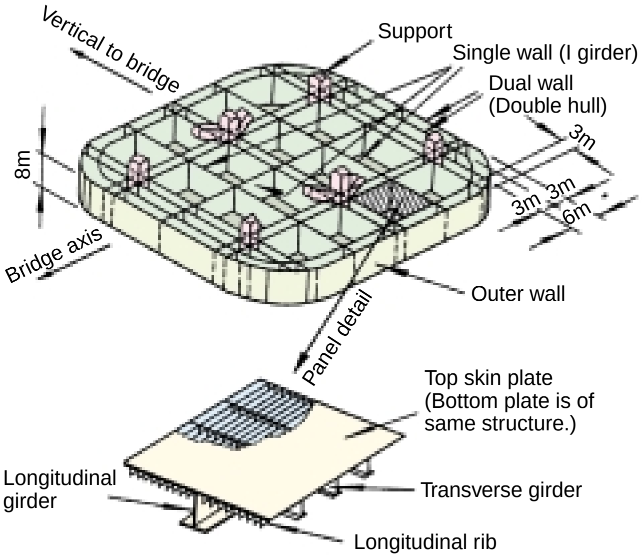


Fig. 11 Structure of Pontoon

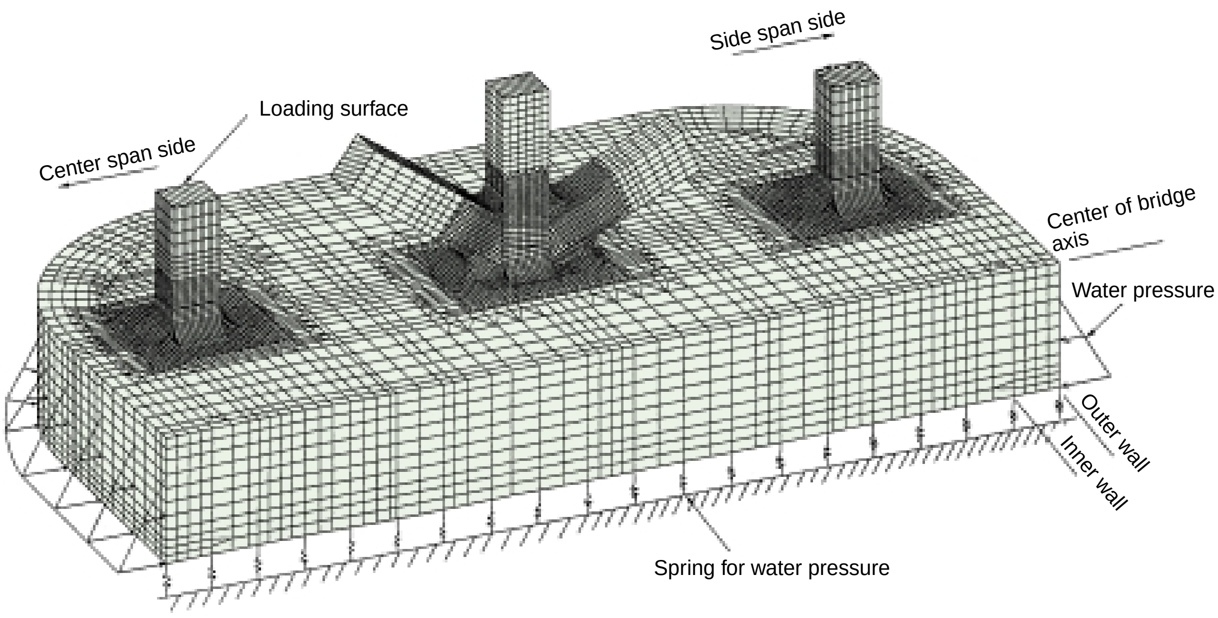


Fig. 12 FEM Model of Pontoon

12. Safety against Ship Collision

Ship collision with a floating pontoon was simulated to confirm safety. Dynamic 3D FEM model analysis (LS-DYNA3D) was used. Fig. 13 shows the FEM model used for this analysis[13)].

Fig. 14 shows the analysis results. The maximum outer wall deformation was approximately 1.7 m, and the deformed outer wall did not reach the water-tight inner wall 3 m inward. This proves that even if the outer wall is partly damaged, water will not enter the inner wall, so traffic on the bridge will not be affected.

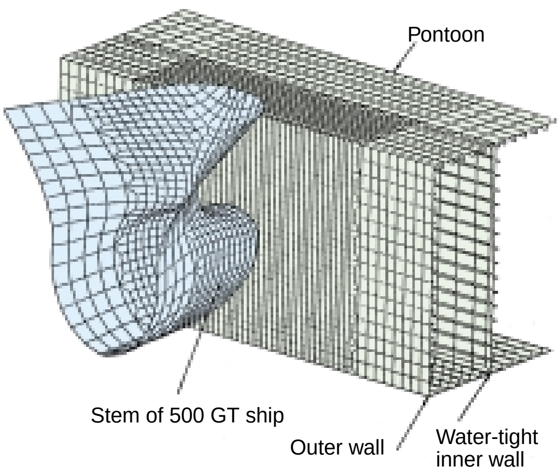


Fig. 13 Ship Collision FEM Model

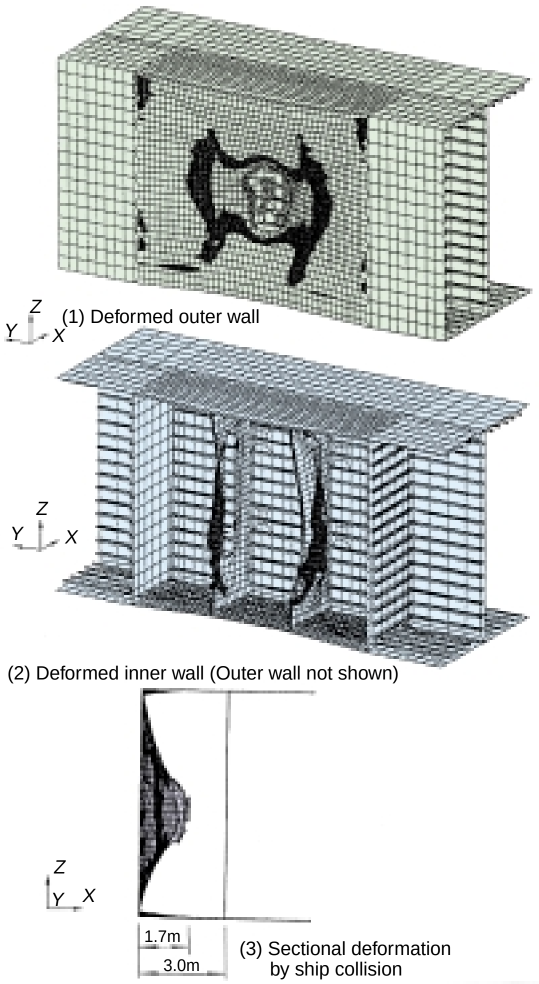


Fig. 14 Results of Ship Collision FEM Analysis

13. Construction Outline & Procedures

The superstructure of the floating bridge with two pontoons was constructed at a dockyard about 10 km away from the bridge installation site. Construction began in March 1998 and was completed in July of 2000. The dock, which measures 62 m wide by 408.3 m long by 12 m deep, could precisely accommodate the two pontoons, with the superstructure girder end protruding by about 5 m outside the dock.

Each superstructure block (average weight: 60 t, maximum weight: approx. 110 t) was mounted by the temporary support method, using two 120 t suspension jib cranes installed on both sides of the dock. Fig. 15 shows the construction procedure (see Photos 6 through 9).

The temporary bents used in constructing the superstructure can be grouped into three types: the center- and side-span stiffening girder supporting bents, which are set on the foundation installed at the dock bottom; center arch supporting bents, which are set on the main stiffening girder structure; and bents for supporting arch members and stiffening girder blocks on the pontoon. The tallest bent used was 36 m tall. The total weight of all temporary bents used was about 4,500 t.

While center-span stiffening girders and structural members were constructed over each pontoon, the pontoons are exposed to sunlight and can warp excessively due to the temperature differential between the upper and lower sides. Before construction, therefore, dimensional measurements were taken during day and night, and adjustments were determined, to secure construction accuracy. Upon installation at the site, the center-span stiffening girders between the two pontoons are also affected by temperature fluctuation. Therefore, the two pontoons were set about 150 mm further apart from each other than the design distance, and center-span stiffening girder blocks were arranged from each end toward the center. After the final block was installed, the Yumeshima-side pontoon was set forward by about 200 mm (150 mm + 50 mm for contraction during winter) to join the stiffening girders.

Photo 6 Pontoons Installed

Photo 7 Superstructure Construction Started

Photo 8 Stiffening Girder Assembled

Photo 9 Full View of the Completed Superstructure

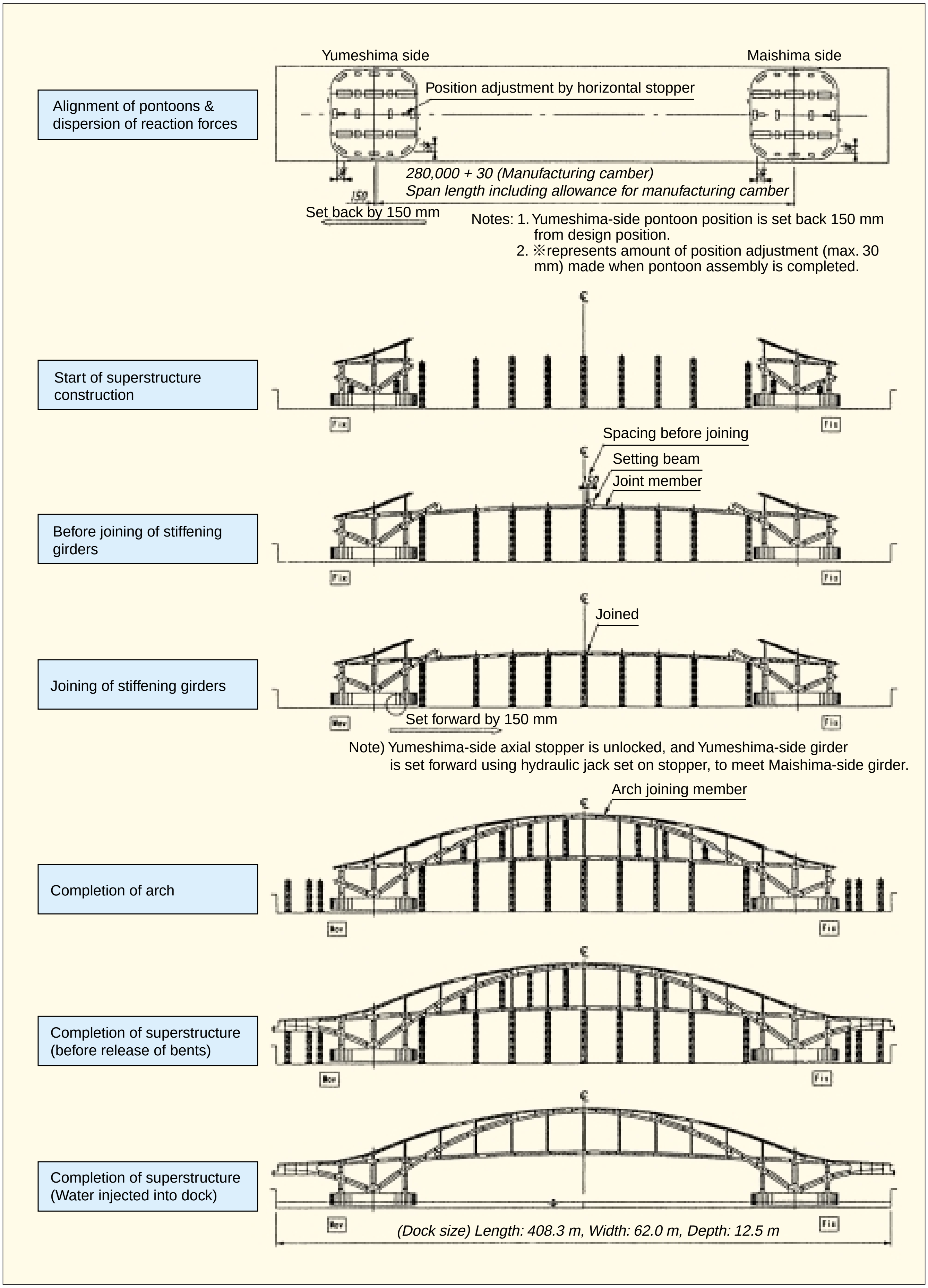


Fig. 15 Procedure for Construction of Pontoons and Superstructure at Dockyard

14. Pulling out of Dock, Towing & Installation

Upon completion, the floating bridge was pulled out of the dock and towed by tugboats to the installation site, where it was successfully installed in mid July 2000. Fig. 16 outlines these operations (see Photos 10 through 12).

(1) Pulling out of the Dock

The floating bridge has two pontoons spaced 280 m apart. After either of the pontoons came out of the dock, the most demanding task was to control the positions of the two pontoons. The position of the trailing pontoon was controlled by operating the dockyard winches and carriages connected to the winch on the pontoon. The position of the leading pontoon was controlled by operating tugboats on both sides of the floating bridge.

(2) Towing

The floating bridge, thus pulled out of the dock, was towed at a speed of about 3 knots by a formation of eight 3,600 HP tugboats, over the 9 miles to the installation site. The towing took about 3.5 hours.

(3) Installation on the Site

At the installation site, the floating bridge was wired to anchors previously installed in the water and on the grounds of Yumeshima and Maishima, by operating the pontoon winch used at the time of pulling the floating bridge out of the dock. The bridge end on the Maishima side was drawn to the mooring system, and a pivot pin was inserted in the same way as in opening/closing the bridge. The other end of the bridge was then rotated by tugboats and connected to the mooring system on the Yumeshima side. Finally, reaction walls were raised to complete the installation. The entire operation, from pulling the floating bridge out of the dock to the on-site installation, was completed in one day.

Closing

The Yumemai Bridge is the world first floating swing bridge. In designing this bridge, therefore, it was necessary not only to meet various existing design standards, such as the “Highway Bridge Specifications,” but also to solve many technical problems. The bridge has successfully been installed on the site, thanks to cooperation from the academic sector, and from various industrial fields, including the shipbuilding, machinery and electric industries. The Yumemai Bridge is scheduled to be fully completed by the late fall of 2000.

Photo 10 Floating Bridge Is Being Pulled out of the Dock

Photo 11 Floating Bridge Is Being Towed

Photo 12 Floating Bridge Is Being Installed at the Site

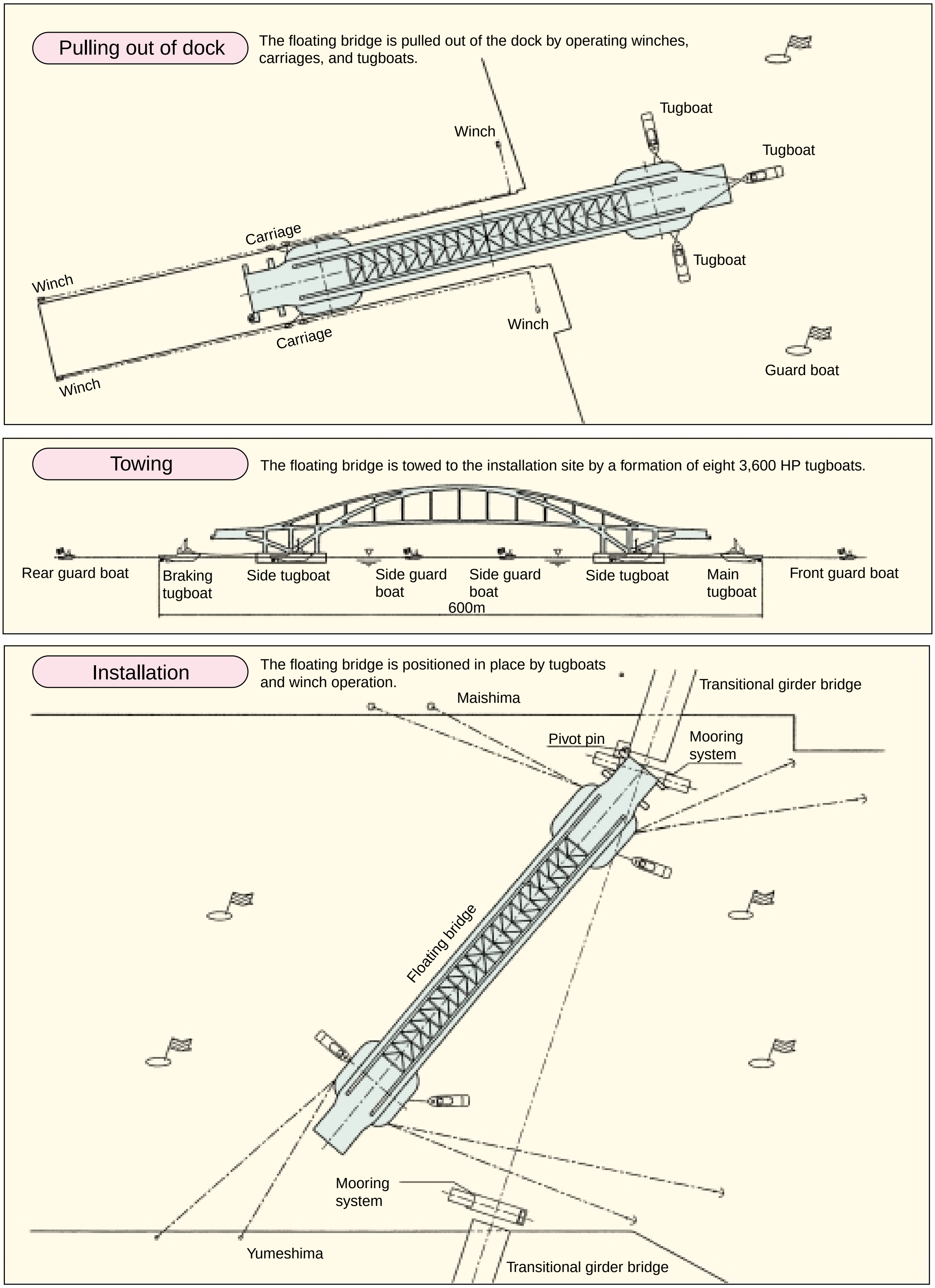


Fig. 16 Pulling out of Dock, Towing & Installation

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