Current Topics of Steel Structures in Tokyo Bay Area

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ABSTRACT

Tokyo bay area as shown in Fig.1 is well utilized and demand of new transportation facilities is very high because of congested air port and roads. Expansion project of a new jacket type runway and new steel bridges crossing navigation channel now under construction are introduced at Tokyo Bay area in this paper. These two projects are applied by new technologies and materials for steel facilities.

1. D-runway project at Tokyo International Airport

1.1 Outline of D-Runway Plan

Tokyo International Airport has already reached critical limit. 65 million passengers per year recently utilized the airport This airport has already three run ways. Another new run way is demanded to cope with the congestion of the air port. A drawing of D-Runway in the current expansion project is shown in Fig. 2. The total length of the runway is 2,500 m and total length of the runway island is 3,120 m. Expansion project ha s strict problems on existing navigation channel, existing airport and Tama river. The construction site lies at the mouth of the Tama River, it is required that the structure be designed to ensure water f low. Total project budget is about 6800 million dollars.

1.2 Current State of Construction of D-runway

Usually off shore air ports in Japan were built by reclamation method. For the construction of D-Runway, the combined reclamation-pier method was adopted. The reclamation method has successful applications at Kansai airport and Chubu International air port. Because construction site is located at a mouth of Tama river, the pier method will not impede the flow of the Tama River.

The major basic facilities to be constructed are the 2,500 m-long runway and taxiway, the access taxiway connecting the runway to the existing airport, an approach lighting bridge, and safety and auxiliary facilities.

As the current project is large in scale and is to be undertaken according to a strict work schedule while the airport remains in service, the work started to meet the schedule of operations at the end of October 2010. Currently, improvement of the revetment ground by sand compaction has been completed and construction of the revetments and reclaimed areas is to follow. Also, pile driving of the jacket foundation and the manufacture and installation of the jackets are proceeding at a steady pace as shown in Photo 1.

1.3 Outline of Jacket Structure

The pier section of the new runway island is located on the Tama River side of D-Runway.

It is about 520 m wide, about 1,100 m long, and has an area of about 520,000 m^2. It is necessary to consider the effect of runway construction on river flow. The water depth on the offshore side of the island is A.P. –19m, the deepest point at the construction site. The water depth is shallower on the side towards the existing airport and is A.P. –14 m where the access taxiway is to be installed. The surface layer of soil is about 20 m thick and consists of soft clay, while the bearing stratum of hard sand or gravel extends to a depth of A.P. –80 m. The adopted pier structure as shown in Fig.3 is designed with concrete floor slabs supported by a jacket structure consisting of a steel girder upper structure, a steel pipe truss lower structure, and foundation piles. Pre-cast concrete slabs are positioned at the center of the floor of about 310000m^2. Total number of slabs is about 107000.

Each PC slab has 6.6mx3.3m with 25tf. UFC slabs are installed at the periphery of main run way. Each slab has 7.8mx3.6m with 10 tf. Total number of UFC slab is about 6900.UFC is ultra high strength fiber-reinforced concrete with more than 180N/mm^2 compressive strength. This slab is lighter than ordinary RC slab. Weight is almost half of the RC slab. Photo 2 shows steel arrangement of the upper PC slab and Photo3 shows completed slab at the factory.

1.4 Design key points of Jacket structure

1.4.1 Live load caused by repeated takeoffs and landings and fatigue strength

The maximum active design load of the airplane is an Airbus A380 with a maximum takeoff weight of 400 tons. The live load with impact factor of 0.4 at landing and 0.3 at take-off to dead load ratio exceeds that of general highway bridges. Accordingly, the upper girder structure must be able to control operational deflection and further provide 100-year fatigue strength. Maximum deflection of the slab is estimated to be 6mm and deflection ratio is 1/2500. Hot spot stress at welding part is estimated by FEM to estimate stress amplitude for fatigue strength of welding part..

1.4.2 Temperature changes on huge pier structure

As to the structural continuity of the pier structure of 840mx520m, the entire pier area should be designed to closely integrate the upper and lower structures as shown in Fig.4.

Mean temperature is 20 ℃ and minimum temperature is -10℃ and maximum

temperature is 40℃. Calculated horizontal movement of the pier is about 150mm.As a

result, the pier is functionally a statically indeterminate structure, which demands that temperature-induced stress in the structure be appropriately assessed and that appropriate structural countermeasures be provided. Sectional stress due to temperature change, especially welded part at the top of the pile is estimated by Finite Element Method (FEM) as shown in Fig.5

1.4.3 Securing seismic resistance of pier structure built on soft ground

As a design condition of the new runway island, two levels of seismic motion were determined—Level 1 seismic motion with a maximum foundation acceleration of 350 Gal, Level 2 seismic motion of 390 Gal, and seismic motion in the scenario earthquake of 487 Gal. The resistance (strength and deformation capacity) of the structural members to these design seismic motions should be secured for a pier structure located on soft ground.

Dynamic response analysis by FEM is adopted to estimate the seismic performance of the pier

1.4.4 Maintenance based on a designed 100-year service life

Because the pier structure is located offshore, a severely corrosive environment, it is important to suppress maintenance and life-cycle costs by providing appropriate corrosion-protection measures and maintenance.

1.5 Performance of Jacket structure

1.5.1 Pile Arrangement

Table 1 shows steel material for pier structure. The interval between piles driven perpendicular to the longitudinal direction of the runway is 15 m. The total number of piles to be driven in the entire pier area is 1,165, and the pile diameter was set mainly at 1,600 mm to secure the necessary bearing capacity. Straight piles were selected and an installation method was adopted that first drives the piles and then covers them with a shop-manufactured jacket structure. Conventional procedure of jacket installation is that the jacket is first installed and then piles are driven into the ground through the jacket legs. However, because the construction site is subject to height limitations due to flight route, pre-driving of the piles is more advantageous.

1.5.2 Upper Girder Structure

The upper girder structure as shown in Photo 4 was arranged in a lattice pattern in which the girders are joined to the jacket legs, and the space between the girders was set at 3,750mm×7,850 mm in order to support the floor slabs above. Two girder heights, 2,500mm and 2,000 mm, were selected depending on the working load, and the I-section girders were selected with a view to securing fatigue strength, manufacturing efficiency, and maintenance. The cover-plates are installed under girders as a countermeasure to prevent corrosion of the steel girders, and the corrosive environment is mitigated by controlling the humidity in the girder space so that maintenance costs can be lessened.

1.5.3 Lower Jacket Structure

In order to secure river function, the stiffening members (braces) of the jacket are laid out at A.P. –4.5 m or under, and moment frames without the use of stiffening braces are adopted near the surface of the water. While this lower jacket structure as shown in Photo 5 is different from conventional jacket structures in which trusses are arranged to the top-level height to improve structural rigidity, as stated below, this lower jacket system is effective and valid in decreasing temperature stress and seismic response and is also effective in reducing areas that need corrosion-protection in the severely corrosive tidal and splash zones.

1.5.4 Installation Unit of Jackets

While efforts were made to reduce the installation number by using larger blocks as much as possible, the standard size of the shop-manufactured jackets was set at 63m× 45 m (six leg arrangement) in accordance with the restrictions imposed on the available derrick vessels and transport barges. The total number of shop-manufactured jackets for the pier section is 198, and the maximum lifting weight per jacket is about 1,650 tons (including auxiliary equipment).Each jacket as shown in Photo 6 installed at the site is integrated by the on-site joining of the steel girders.

1.6 Joint part between jacket structure and reclamation

The runway is divided into steel pier part and reclamation part. Ground relative settlement at the joint part is influenced by reclamation part. Total about 8m of settlement is predicted at reclamation part and 20% of it is also predicted after the construction. Large value of relative vertical settlement is predicted. Also Horizontal displacement at the joint part is calculated during earthquakes. To cope with such large value of ground movement, steel sheet pile revetment type is adopted. Fig .6 shows outline of the revetment structure. Double wall by steel sheet pipe pile are driven into the ground and reinforced concrete slab is cast at top of the piles. Wave dissipated member is installed at the front of the revetment. The connection slab is also installed between revetment and the jacket. To absorbed differential movement, flexible joint as shown in photo 7 is installed. The maximum absorbed horizontal displacement is about 60cm.

1.7 Corrosion-protection Design

1.7.1 Selection of Corrosion-protection Method

The pier section is covered with large floor slabs and is installed in a severely corrosive environment where airborne sea salts adhere to the structure and are not cleaned by rainfall. Further, because the structure lies offshore, maintenance is not easy. To cope with such severe circumstances, new corrosion-protection methods for steel products are selected that can secure long-term durability.

1.7.2 Seawater-resistant Stainless Steel Lining

The selected corrosion-protection method as shown in Photo 8 sheathes the surfaces of structural steel products with a lining of seawater-resistant stainless steel offering excellent corrosion-protection in offshore environments. The method is superior in impact resistance and wear resistance to other corrosion-protection lining methods.

Further, because this method offers greater long-term durability than conventional heavy-duty corrosion-protection methods, its use in the construction of port and harbor structures has increased in recent years.

The stainless steel that was adopted is SUS312L. This product is known as a super stainless steel with improved pitting and crevice corrosion resistance made possible primarily by increasing the content of chromium and molybdenum over general stainless steel (SUS304, SUS316L).The thickness of the plates applied in the general pier section is 0.4 mm. Because lining is difficult by means of general TIG welding, combined indirect seam and plasma welding is being applied.

1.7.3 Titanium Cover Plates

Titanium cover plates have two functions. First is to mitigate the corrosive environment in the extensive area encompassing the upper steel girders where corrosion protection is required and second is the scaffolding used for pier maintenance. These plates cover the lower surfaces and the peripheral sides of the upper steel girders. The adopted cover plates consist of titanium panels with on outer skin of highly corrosion-resistant titanium sheets with 0.35mm, a core of polyisocyanurate materials with 35mm thick, and an inner skin of steel sheets with 0.6mm thick. The titanium panels are suspended so that they cover the steel girders using light-gauge steel shapes called fasteners. The titanium panels measure 1 m in width and 11 m in length and the space between panels is composed of the joining structure.

1.7.4 Humidity Control inside Girders

The corrosive environment in the space between the girders and the cover plates is greatly improved by the cover plate system, but it is forecasted that the coating inside the girders will deteriorate due to condensation caused by air penetrating from the outside and by temperature change. In order to prevent condensation, a corrosion-protection method was adopted that utilizes a dehumidifier system consisting of dehumidifiers, circulation fans, air-supply ducts, and other equipment to keep the relative humidity of the air surrounding the girders at 50% or lower. The blocks allocated for dehumidifiers were determined by referring to examples of bridge applications; 49 dehumidifiers were arranged in the pier section.

In deciding the specifications for the dehumidifier system, reference was made to meteorological conditions at the construction site and studies were conducted using air-conditioning simulations and other means in order to secure a relative humidity of 50% or lower. Further, air-supply ducts were run throughout the entire pier area and specific devices were installed so that the humidity inside the girder space would be made uniform through the forced supply of dry air discharged from the dehumidifiers.

1.7.5 Cathodic Protection

Cathodic protection of the anodic type, which has many successful examples of reliable use in port and harbor construction, was adopted for the submerged and underground sections of the pier. Based on survey results for seawater quality at the construction site, the current density for the first stage of corrosion protection was set at 130 mA/m^2 for the upper submerged section (A.P. –2 m or higher) and at 100 $mA/\ m^2$ for the other lower submerged sections. The design service life of the anodes is 35 years, and about 16,000 anodes are planned for installation throughout the entire pier area.

Reference

(1)Takatoshi Noguchi,Noriyoshi Suzuki:Tokyo International Airport Re-expansion Project,Steel Construction today & tomorrow,No.22,March,2008,

(2) http://www.pa.ktr.mlit.go.jp/haneda/

Fig.1 Tokyo Bay Area

Fig.2 Expansion project at Tokyo International Airport

Photo 2 Steel arrangement at slab panel

Photo 3 Fabricated slab panel

Photo 1 Construction site in 2008

Photo 4 Fabricated upper steel slab

Photo 5 Jacket part

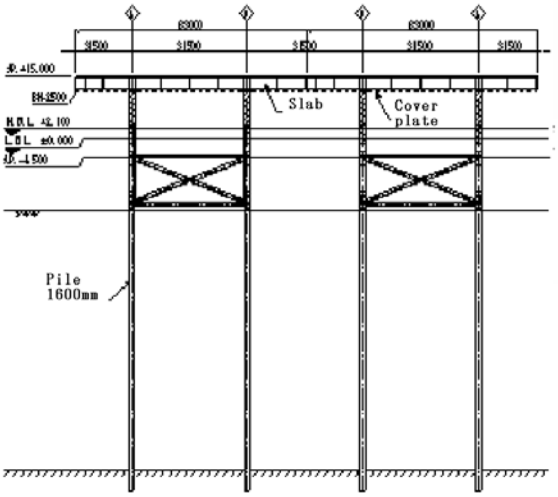


Fig 3 Cross section of pier structure

Photo 6 Installation of jacket part

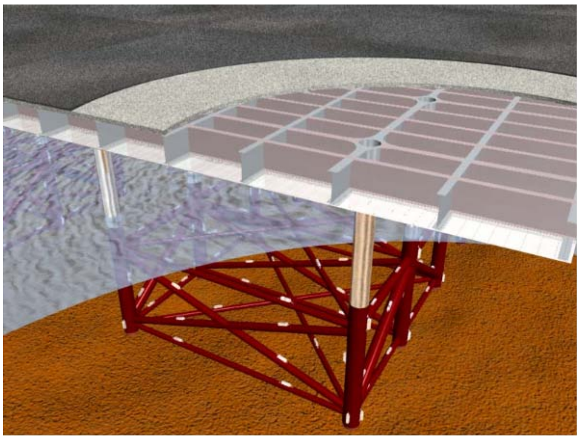


Fig.4 Outline of pier structure

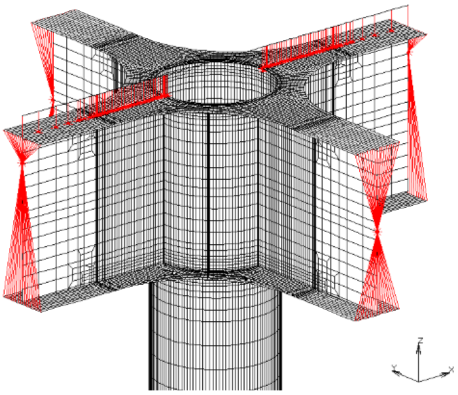


Fig.5 Stress distribution of pile top

Table 1 Steel material for jacket structure

	Specification	Volume
Jackets	45mx63mx32m(height)	198 units
Steel girders	BH2500,BH2000,BH700	160000tf
Substructure	Leg φ1600~φ1960 Braceφ700~φ1422.4	90000tf
Foundation pile	φ1600 φ1422.5 x85~90m φ1320.8	1165 piles 90000tf

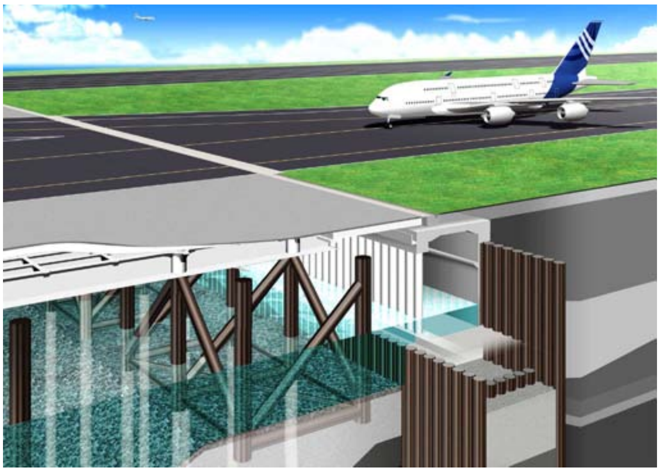


Fig 6 Connection part

Photo.7 Model test of flexible joint

Photo 8 View of piles covered by stainless steel

2. Tokyo Port Bayside Bridge

2.1 Introduction

The coastal area of Tokyo Bay surrounding the Port of Tokyo is a place of a high concentration of not only port and harbor logistics functions, but also the nation's major production, commercial, business and urban functions. Owing to all the international trade cargo handled in the area and because of the urban activities conducted in the surrounding metropolitan area, traffic volume is immense. Due to increases in vehicular traffic, road conditions have deteriorated, thereby causing an increase in the lead time required to transport international container cargo from port and harbor facilities to production and consumption sites. In the Port of Tokyo, a strategic plan is currently underway that is designed to enhance smooth inter-city logistics for the transport of international trade cargo in the coastal area. Known as the Tokyo Port Bayside Roadway, this thoroughfare will extend 8 km. The first phase of construction began in July 1993 with a 3.4 km-long underwater tunnel joining Jonanjima with a reclaimed site outside the Central Breakwater; this phase ended in April 2002 when the tunnel was put into service. Currently underway, the second phase of the plan calls for the construction of a 4.6 km-long road that will include the 2.9 km-long Tokyo Port Bayside Bridge (provisional name) as shown Fig.7. The road is scheduled for completion in 2010. Total budget is about 1400milliom dollars.

2.2 Outline of construction

The following are noteworthy features of the construction site for the Tokyo Port Bay Side Bridge.

1) Because the bearing strata are soft, installation of the foundation structure will require greater than normal depth.
2) The bridge will have a long span because it will extend over the third fairway with 310m width at the Port of Tokyo.
3) The bridge is subject to restrictions imposed for being within the flight area(height is 97m) of the Tokyo International Airport.

In order to meet these restrictions and to secure the required bridge performances, a variety of new bridge design technologies have been incorporated at every turn of the construction. Fig.8 shows outline of the truss box type bridge. The above-water substructure consists of nine piers. The two main piers (MP2 and MP3) that sandwich the third fairway of the Port of Tokyo are solid wall-type piers, and the substructure installed between the side spans uses hollow reinforced-concrete piers.

At the bridge construction site, there is a thick layer of soft alluvial clay (N value ≒ 0), a gravel layer located below A.P. –75m on the Central Breakwater side that serves as the bearing stratum of the foundations, and a sand layer located below A.P. –50m on the Wakasu side that serves as the bearing stratum of the bridge piers. Both bearing strata are located at relatively great depths.

Because of the highly demanding ground conditions under the bridge structure and, further, because the bridge foundation must demonstrate sufficient supporting capacity and stability against design seismic motions throughout the 100-year service life of the bridge, studies were conducted on a variety of structural types as they apply to foundations. As a result, the steel pipe sheet pile foundation was selected as the most suitable structure.

2.3 Structural Outline of Bridge Foundation

The following performances are required of the bridge foundation as shown in Fig.9.

1) High rigidity of pile foundation structure due to the softness of the ground at the construction site

2) High work safety and less environmental burden made possible by reducing soil excavation during construction

3) High construction efficiency and cost advantages

Steel pipe sheet pile foundations were adopted as the structural type that meets these requirements with a high degree of satisfaction as shown in Photo 9. Steel pipe with a diameter of 1,500 mm was selected in consideration of the drilling depth, residual stress control and supporting capacity. Photo 10 shows driving pile by hammer. In order to ensure shear strength of the steel pipe sheet piles during an earthquake, interlocking joints were adopted that use checkered steel plates in combination with high strength mortar as shown in Photo 11. Photo 12 shows concrete casting on the foundation for the pier and Photo 13 shows completed main pier.

According to seismic design, large rubber bearing shoes is installed between the super structure and the pier as shown in Fig.10 and the detail of the rubber bearing shoe is shown in Fig 11.

2.4 Structural Features: Weld Joints

As stated above, most of the bridge’s structural members are being joined by welding, where the efficient joining method most rarely found in conventional bridge construction has been adopted. In the conventional joining method applied to sections where chords and other members that compose the truss are concentrated, the joint structure comprises members that are individually joined via a splice plate. In the current bridge, the conventional method is replaced by a joint structure in which each of the members concentrated at the panel point is directly joined to the others to form a more compact panel point as shown in Photo 14. In this structure, optimization of the member cross sections is enabled by the direct transfer of stress to each member. Because the thin crossed axes angle is liable to generate in the compact panel point, advanced welding technology and strict weld control are required in the welding operations. Detail of truss joint is determined by FEM as shown in Fig.12 and Fig. 13.

2.5 Bridge Superstructure

The Tokyo Port Bayside Bridge consists of 5 blocks differentiated by location: two land-based approach sections (both ends), two offshore approach sections, and the main bridge section. Of these, the block that will be the largest in scale and require the most difficult construction work is the main bridge section—a three span continuous truss-box composite structure having a total length of 760 m and a center span of 440 m. All the other land based and offshore approach sections are multi-span continuous steel slab box girder structures.

Structural Features: Adoption of High-performance Steel for Bridge Construction

A distinctive feature in the construction of this bridge is the use of Bridge High Performance Steel (high-performance steel for bridge construction) to reduce the bridge's total weight as much as possible. Such use is a first in long-span bridge construction in Japan. The Bridge High Performance Steel (BHS) is a high-performance material as shown in Fig.14. This advanced steel not only has higher strength than conventional steel but also offers excellent weldability and workability as shown in Table 2.

Recently, reduced construction expenses, improved durability and lower maintenance costs are being called for in social infrastructure development. To meet these requirements as they apply to bridge construction, there is a growing need for steel products with higher performances; such high-performance steel is being used extensively to enhance weight reduction in the construction of the Tokyo Port Bayside Bridge.

Reference

(1) http://www.pa.ktr.mlit.go.jp/tokyo/

Fig 7 Design of bridge

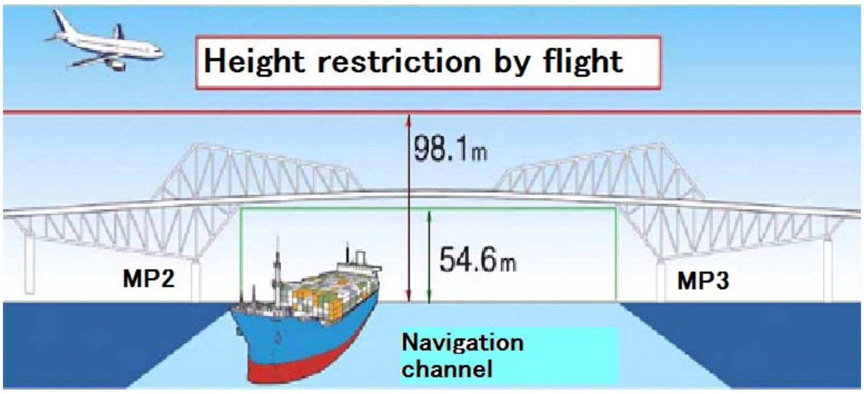


Fig.8 Outline of the bridge

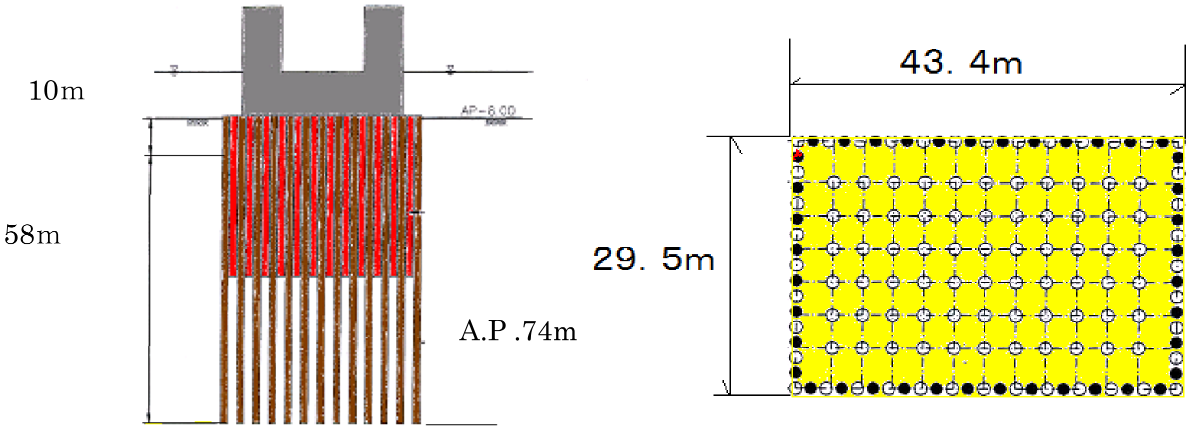


Fig.9 Outline of the foundation

Photo 9 Foundation

Photo 10 Driving pile by hammer

Photo 11 Joint of the piles

Photo 12 Pier on the foundation

Photo13 Completed main pier

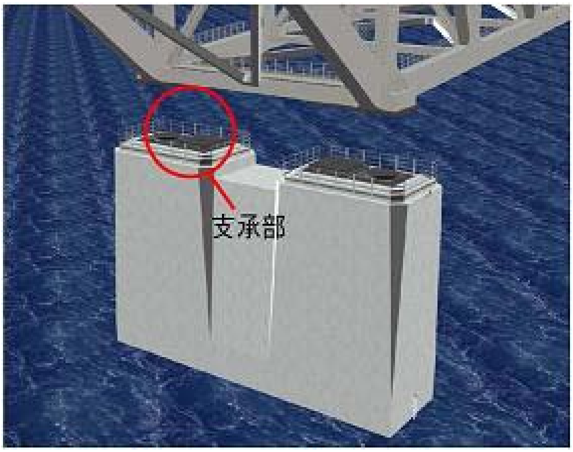


Fig.10 Rubber bearing shoe

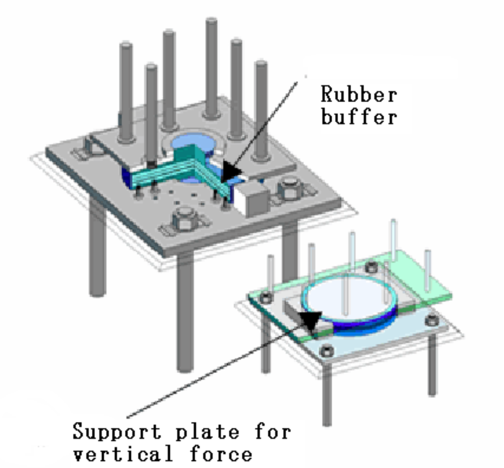


Fig.11 Details of bearing shoe

Photo 14 Truss joint part by welding

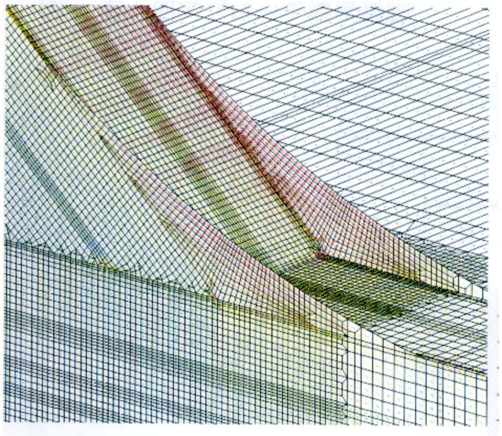


Fig.12 FEM model of truss joint

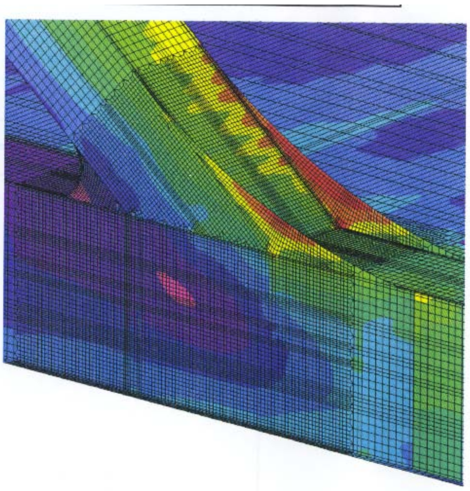


Fig.13 Stress distribution of the joint

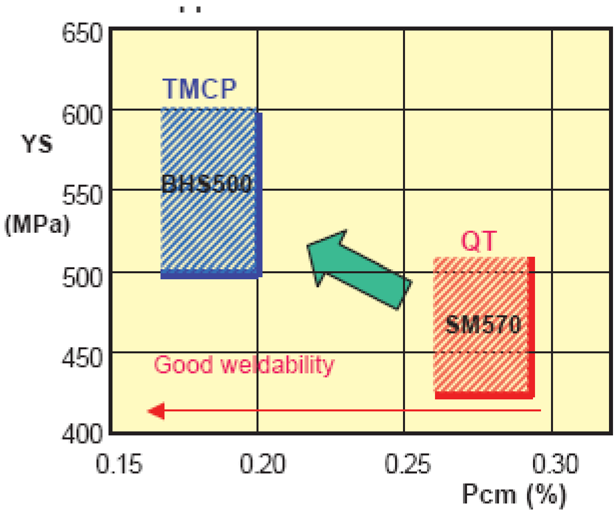
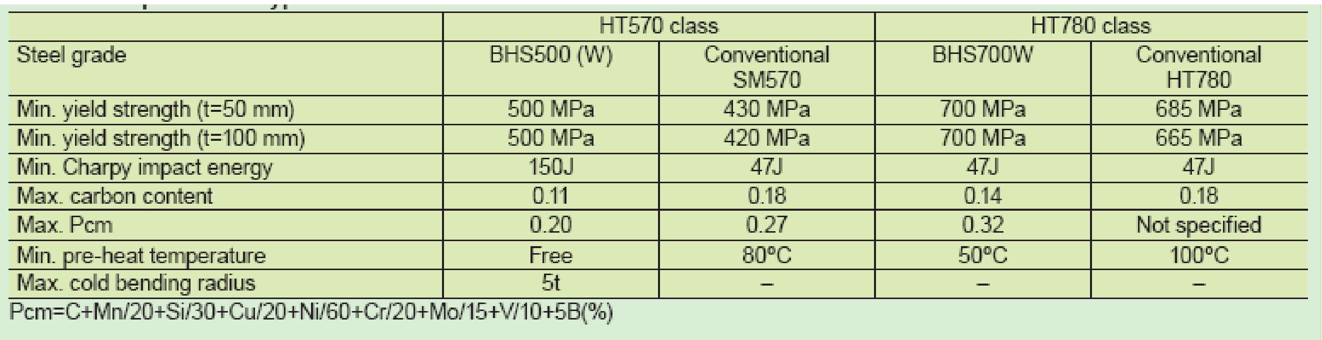


Fig.14 High performance steel and ordinary steel

Table 2 Properties of high performance steel



	HT570 class		HT780 class	
Steel grade	BHS500 (W)	Conventional SM570	BHS700W	Conventional HT780
Min. yield strength (t=50 mm)	500 MPa	430 MPa	700 MPa	685 MPa
Min. yield strength (t=100 mm)	500 MPa	420 MPa	700 MPa	665 MPa
Min. Charpy impact energy	150J	47J	47J	47J
Max. carbon content	0.11	0.18	0.14	0.18
Max. Pcm	0.20	0.27	0.32	Not specified
Min. pre-heat temperature	Free	80°C	50°C	100°C
Max. cold bending radius	5t	–	–	–

Pcm=C+Mn/20+Si/30+Cu/20+Ni/60+Cr/20+Mo/15+V/10+5B(%)