Advancement of urban functions and “formation of an urban space that makes the most of history and culture” are being promoted in Tokyo's Marunouchi district with an aim of Tokyo being a “world-leading international city that is attractive and lively.” Tokyo Station Marunouchi Building bears a particularly important role in railway architectural history, presenting a “face” for the capitol of Tokyo. Its cultural value has been recognized, and it was designated an important cultural property of Japan in May 30, 2003. Construction started in 2007 to preserve and restore† the building as much as possible to the form it took when initially built, and the restored station building opened in October 2012.

1 Introduction

Advancement of urban functions and “formation of an urban space that makes the most of history and culture” are being promoted in Tokyo's Marunouchi district with an aim of Tokyo being a “world-leading international city that is attractive and lively.” Tokyo Station Marunouchi Building bears a particularly important role in railway architectural history, presenting a “face” for the capitol of Tokyo. Its cultural value has been recognized, and it was designated an important cultural property of Japan in May 30, 2003. Construction started in 2007 to preserve and restore† the building as much as possible to the form it took when initially built, and the restored station building opened in October 2012.

2 History of Tokyo Station Marunouchi Building

2.1 Opening

Tokyo Station Marunouchi Building, affectionately known as the “red brick station building,” was designed by Dr. Kingo Tatsuno, one of early modern Japan's most prominent architects. It took a long time to bring about this central station for Tokyo due to the impact of factors such as the Russo-Japanese War and design changes. Construction started in March 1908, and it was completed six years later on December 20, 1914, opening as “Tokyo Station.”

2.2 Original Form

The station building is a long structure facing the Imperial Palace, spanning approx. 335 m in a north-south direction. When originally constructed, it was three stories high with one underground level. Its back side (platform side) had a single-story attachment, and total floor area was about 10,500 m². Originally, the south dome entrance was for departing passengers and the north dome entrance for arriving passengers, the central part was reserved for use by the Imperial family, and north of that was an exit exclusively for passengers of electric trains. The first floor housed facilities such as the stationmaster's office, a resting and waiting room for the Imperial family, first to third class waiting rooms, a restaurant, and a luggage office. The second and third floors had hotel guest rooms and restaurants on the southern half and Government Railway offices on the northern half.

2.3 Earthquake and War

The station building did not suffer major damage in the 1923 Great Kanto Earthquake, but fire from bombing on May 25, 1945 towards the end of World War II damaged the roof and ceiling. In recovery work, Kaoru Takayama, who was transferred from the Army to the Building Division of the Ministry of Railways, made structural design of wooden trusses that were composed of rectangular wooden planks combined and joined with dowels and nails. For the roofing material, galvanized plates were installed and finished by painting. The recovery work was completed in March 1947, with the original three-story structure changed to two stories.

From 1951 to 1952, the single-story part (gabled roof) was completely reroofed with natural slate from Ogatsu in Miyagi Prefecture, and the north, south, and central domes were reroofed with slate from Tome in Miyagi Prefecture. The interior ceiling was steel frame-backed duralumin boarding finished with paint as duralumin used in airplanes became easier to acquire with the war ending.

3 Investigation of Original Construction Conditions and Current Framework

Documents on the structure of the Tokyo Station Marunouchi Building included brick test reports and investigation reports on the building’s foundation pine piles. Of those, the Tokyo station construction report† by Hikozaburo Kanei, who was a Government Railways construction supervisor at the time, covered in detail the building and construction conditions at the time the station was originally built. In 1989, a committee to study the structure of the Tokyo Station Marunouchi Building headed by University of Tokyo honorary professor Tsuneo Okada also was set up at the Japan Building Disaster Prevention
Association. The following is an overview of the structure based on a report by that committee as well as the result of later on-site investigations.

3.1 Foundation Structure
The ground near the station building is soft, but the ground that it is built on is far better than the surrounding ground. Fig. 2 shows the details of the foundation of the gable-roofed part. The foundation of the gable-roofed part and partitions is 3.8 m lower than the ground surface, and its pine piles with 212 mm end diameter and 5.5 to 7.3 m length were constructed in approx. 0.5 m intervals using steam-powered pile drivers. The pine logs used for the piles were purchased from the Aomori forestry office. Concrete slabs 1.2 mm thick were constructed on top of the piles, and on top of those, two layers of granite 1.2 m or 0.7 m to a side and 0.3 m thick were laid at the position where steel frame columns stand, with brick laid between the building stones. The dome parts have the same structural type as the gable-roofed part, including the foundation type, with pine piles driven at approx. 0.6 m intervals.

3.2 Steel Frame Structure
3.2.1 Framework Structure
Fig. 3 shows part of the framework structure of the public part. Much of the materials used for column members are 10-inch I beams. In corner parts or locations with large load, built-up columns are used as shown in Fig. 4. Column base parts are all the same height, regardless of whether or not there is basement space, and they are fixed to the foundation stone by bolts. This was done to be able to build basement space as needed with just a little extra work.

3.2.2 Roof Trussing
Public part roof trussing has, as shown in Fig. 5 (1), a 20 m span and 30° pitch laid in 4 m intervals with the trusses connected by four steel angles that substitute purlins. Wooden members are used for all other purlins. The upper roof of the central Imperial family room is 1.8 m higher than the public part, and it has a pitch of 45°. The shape is shown in Fig. 5 (2). The upper dome roof of the octagonal hall of the left and right entrances has eight arched trusses in 20 m spans with height of 6.9 m installed side by side as shown in Fig. 5 (3). Eight arches converge at the center.

3.2.3 Steel Frame Quality
A total of 3,135 tons of steel frame is used in the individual structures, and the policy was to use domestically procured materials as much as possible. While 10-inch I beams or 8-inch channels used for much of the column members and horizontal tying members could be made at Yahata Steel Works, their cross-sectional shape tended to be large, so imported material was used. Other I beams, channels, L beams, flat iron, iron plates, round bars, and the like used were made at Yahata Steel Works. Of the material used, 56% was domestic, and 44% imported from Carnegie in the USA and Frodingham Iron and Steel in Great Britain.

When purchasing steel frame material, regulations were prescribed for unit strength tests and bending tests including for steel bar used in rivets, and inspections were performed. Also, member length too was controlled, with regulation such as rolled steel frame cross-sectional area tolerance being set to within 2.5%. The imported materials underwent detailed testing inspection by foreign engineers employed by Government Railways, and materials from Yahata Steel Works were tested at the works, with only items passing tests being used.

3.2.4 Steel Frame Production and Erection
Production and assembly of the steel frame was done at Ishikawajima Shipyard from September 1909. The entire
building was separated into 10 construction sections, and production was done with sequentially by section starting from the south end. After temporary assembly in the factory after each completion of sections, erection work started on August 1, 1910.

A steam-powered crane was produced for erecting the steel frame. That crane moved on tracks, and it was able to lift loads of 7 tons 21 meters and rotate. Using this method, the public part and the framework of each of the spires could be constructed, but the round roofs of the north and south octagonal spires were 39 m at their highest part, so they could not be constructed by this machine. They were constructed by setting the crane on other columns after they were erected separately. Using these methods, erection of the complete steel frame was completed in September 1911. It only took two years from the start of steel frame production and one year and one month from start of erection at the worksite.

3.2.5 Investigation of Existing Steel Frame
In an attempt to reuse the steel frame, which suffered fire damage in the bombings, strength tests (tension tests) and weldability confirmation were conducted on steel frame columns extracted from the roof level steel frame of the north dome. As a result, it was found that strength was almost the same as the original unit strength of 498 N/mm², equivalent to structural rolled steel. From the results of weldability confirmation (component tests, macroscopic tests, and sulfur print tests), it was found that the steel frame had a high sulfur component content, and lamellar tearing had to be taken into account when applying welds. And from microstructure photographs, it was found that the microstructure of metal was same as that in ordinary rolled steel and that the steel frame structure was not exposed to a temperature (750 °C) at which steel is degraded. Appropriately reinforcing material was added to fire-damaged steel frames as needed, and attempts were made to preserve and utilize those as much as possible.

3.3 Brick Wall and Floor Structure
3.3.1 Structural Brick
Records note that, at the parts where brick wall contacts the steel frame, all bricks were cut and laid so as to be in close contact with steel frame, and that, at the part near the horizontal tying members where a half brick was extremely thin, bricks were bound by steel wire. It was also confirmed in on-site investigations that atypical shape bricks were solidly packed in the space between the steel frames in the wall.

3.2.2 Face Brick
Due to the large amount of face brick used for the time, brick factories lacked the experience such volumes, even though they had the equipment to do so. Records thus state that only about 40% of the bricks produced passed inspection.

3.3.3 Concrete Mixed with Brick Fragments
Brick fragments used as aggregate were waste cut-off material generated at the parts where brick contacted the steel frame. On-site investigation confirmed that concrete mixed with brick fragments and steel wire was used on the inner walls of the dome attic and window frame parts.

3.3.4 Floor Structure
The floor was cinder concrete with a thickness of 333.3 mm on the first floor, 181.8 mm on the second and third floors, and 151.5 mm in the attic. Cinder was sifted to a size of 15.2 to 24.2 mm with lime waste in the embers removed. Cinder concrete mixture was 1 part cement, 2.5 parts sand, and 5 parts cinder. On-site investigation identified steel frame joints and the like at wall and floor connection and outer wall and inner wall connection parts matched what was noted in past documentation.

3.3.5 Existing Material Tests
Core samples were taken of concrete mixed with brick fragments and cinder concrete, and compressive strength tests and splitting tensile strength tests were performed on those. As a result, it was determined that cinder concrete was not suitable in terms of quality for continued use, so it was removed and entirely new floor slabs were laid.

Concrete mixed with brick fragments making up the wall was slightly less strong than past records stated, but it was of a level that would not be a problem in terms of load bearing ability. It could be said to be in very good condition for a brick structure more than 75 years old (at the time, in 1989), and it was determined that the brick could continued to be used as a structure.

4 Structural Design for Station Building Restoration

4.1 Design Policy
With “permanent preservation and utilization of the red brick station building” as the theme of the preservation plan, specific target aseismic performance was set. That target was for no cracking to occur in the brick walls in a medium-scale earthquake and for cracking of the brick walls being allowed but the building being usable without major repairs in the event of the largest earthquake anticipated.

The amount of existing brick to be preserved and the framework form of the third floor restored part were studied, and the amount of reinforcement needed to secure target performance was calculated. As a result, it was found that almost no aseismic reinforcement would be needed if a seismic isolation system were adopted, while about half of the estimated inner wall would need aseismic reinforcement with the conventional seismic retrofitting method. Therefore, a decision was made to adopt seismic isolation taking into consideration factors such as safety, flexibility of use, and preservation precision being improved and impact on the box culvert of the Sobu Line underground area.

Taking into account the aforementioned target aseismic performance, targets were set in design. Specifically, those were to not allow existing brick walls of the station building to crack in a large earthquake and for the seismic isolation structure to function in seismic movement 1.5 times that of a large earthquake.
Fig. 7 Design Flow for Preservation and Restoration

(considered to be seismic movement greater than anticipated).

4.2 Placement of Seismic Isolation Members

With the station building, there was a constraint of small horizontal clearance from adjacent structures; so in adopting seismic isolation, a system was needed that could keep response horizontal displacement of the isolating layer small while containing response amplification of the superstructure. For that reason, oil dampers were used extensively along with laminated rubber having lead plugs to increase damping force of the isolating layer, enabling horizontal displacement in an earthquake to be controlled. With factors influencing performance of the seismic isolation system (isolation rubber rigidity, lead damper and oil damper damping force) as parameters, the optimum combination of the isolation system (isolation rubber rigidity, lead damper and oil damper damping force) was selected. Laminated rubber having lead plugs to increase damping force of the isolating layer was considered, and laminated rubber and oil damper performance and placement were decided.

4.3 Response Analysis Results

By using 158 dampers, displacement of the isolation layer could be kept to just 13.7 cm even in a large earthquake. Response acceleration at the first to fourth floor would be about 200 cm/s² and affect on people and furniture small, but that of the steel frame dome part would be up to 900 cm/s², so sufficient consideration is made for dome ceiling and other finishing material attachment. Inter-story deformation angle of the existing brick parts too is 1/1600 or less, which is within the assumed cracking distortion angle (1/5000) of brick walls. Even in seismic motion 1.5 times that of a large earthquake, isolation layer deformation would be 21.7 cm and superstructure inter-

5 On-site Mockup Tests

In order to confirm validity of design assumptions and the actual yield strength of the structure, mockup load tests were performed using existing brick wall at its current location. Tests were made to be as close as possible to actual construction methods and actual proportions. Loads were applied to destruction level in order to obtain the ultimate yield strength, so brick walls scheduled to be removed were selected.

5.1 Out-of-plane Direction Proof Tests

The edge of a 1.0 m wide and 2.5 m high double-stacked brick wall including one encased steel frame column was cut from the surrounding wall and force applied in the out-of-plane direction as shown in Fig. 8. According to design, resistance in the out-of-plane direction is mostly borne by the encased steel frame. Difference between load on the encased steel frame and actual resistivity was evaluated by measuring strain on the encased steel wall column near the foundation. Composite flexural capacity of the encased steel frame and brick wall was confirmed to be 1.5 times that of steel frame alone.

5.2 In-plane Direction Proof Tests

In order to confirm maximum shear capacity that can be borne in the in-plane direction of a brick wall serving as a quake-resistant wall and maximum flexural capacity borne in tensile resistance of encased steel frame, applied force tests were performed on the
test sample shown in Fig. 9 (part of compressed side reinforced by RC reinforcing frame to confirm maximum pullout resistance of encased steel frame). Resistance to shear cracking was greater than assumed in raw material tests (joint shear capacity), and resistance of about 0.33 N/mm² was obtained. This shear capacity includes the reinforcing effect of encased steel frame, and it is assumed to be closer to the actual conditions.

Bending moment of leg parts equivalent to shearing force borne was confirmed by measuring strain of encased steel frame (vertical members) to effectively contribute as tensile force bearing members. In actual construction, existing aseismic performance could be secured in the method of joining new structures by similarly adopting a form where new footing beams are embedded.

6 Construction

In order to build the new underground skeleton while restoring the existing aboveground framework and install seismic isolation members at the boundary between those, construction was done while the aboveground part was supported on temporary support piles. The procedures of this construction method are shown in Fig. 10.

Step 1: Install permanent piles and piles also used for temporary support.

Step 2: Install reinforced concrete vertical beams and tie beams at the brick wall leg part.

Step 3: Temporarily support weight of building on underground permanent steel pile columns and temporary support columns by jacking up.

Step 4: Remove existing foundation after initial excavation, and then build underground skeleton by inverted construction method.

Step 5: After building underground skeleton, move load to permanent framework using flat jack (Fig. 11) positioned at top of the seismic isolation supports, and then remove bottom section (B2 level part) of temporary support columns.

Step 6: Thermal-cut temporary support columns used in horizontal restraints of seismic isolation layer to make the building seismically isolated.

In order to transfer the vertical load from temporary support columns to the underground skeleton, preload was introduced by injecting mortar in the flat jacks on top of the isolators. At that time, work proceeded with a target pressurizing force within a margin of error of about 20% the amount of set pressurizing force (calculated from design axial force). At the same time, measurement control was done to prevent in-plane distortion angle from exceeding the target value of 1/2500 (control limit value of 1/2000) and keep cracking from occurring in the brick skeleton. After introducing preload, shear transfer plates were welded to join the superstructure and isolators and grout filled in the gap.

Transferring load from temporary support piles to 352 seismic isolators took about three months with 10 isolators being done a day. After completely transferring the vertical load in this way, seismic isolation was completed in September 2011 by releasing the horizontal restraints. The remaining temporary support columns and temporary support beams were then cut and removed and
underground interior and equipment work and other finishing work processes started.

Temporary support members (ground level floor beams, temporary support columns and underground skeleton) were designed to have vertical stress when temporarily supported that is smaller than long-term allowable stress. This was done because the north, central and south ticket gates and the Sobu central stairway was used by passengers during construction and members remained temporarily supported for many years before transfer.

The entire aboveground part was in a temporarily supported state when the 2011 Earthquake off the Pacific Coast of Tohoku struck. However, the temporary support members were designed to have less than short-term allowable stress even in an earthquake, so there was no damage and construction could continue.

### 7 Conclusion

Assuring the building's safety as a cultural asset that will continue to be used while preserving the authenticity of the historical building was a condition that had to be met for preservation and restoration of Tokyo Station Marunouchi Building. Moreover, the optimal structural reinforcement design for the station building was one where the historical building would be kept while making use of it as a structure, as opposed to keeping the red brick walls as merely a façade, in looking for a way to make the minimum changes to the building in the process of accurately identifying the structural characteristics of the existing building. Tokyo Station Marunouchi Building was restored to its original form while adding the modern technology of "seismic isolation structures" to the steel frame brick structure, which was cutting edge architectural technology in Japan's early modern Western style architecture.

† JR East uses "restore" to mean “returning parts of the existing structure that had been repaired or modified to their original form.”

### Reference: