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Structural Design of Nakanoshima Festival Tower

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Abstract

Nakanoshima Festival Tower is a 200 m high-rise complex building which contains a renewed 2700-seat capacity concert hall known as “Festival Hall” and offices including headquarter of a news company. In order to build up an office tower on the hall which requires large open space, a giant truss system is employed. The giant trusses being composed of mega-trusses and belt-trusses support all the building weight above them and transfer the load to the outside of the hall. The building also requires high seismic resistance performance for a news company. Application of mid-story seismic isolation enables the building to satisfy high-level seismic resistance criteria.

Keywords: Structural design, Complex building, Mega structure, Steel structure, Mid-story seismic isolation

1. Introduction

The original Festival Hall in Osaka was constructed in 1958. As a hall with history and tradition, it has long been popular as a fountain of culture and art in Osaka. The hall boasted a world-top-class scale of 2,700 seats and was characterized by excellent acoustics referred to as "sound from the heavens". It served 50 times as the venue of The Osaka International Festival, and has been the stage for masterful performances by famous musicians and conductors such as Herbert von Karajan and Leonard Bernstein. However, in December 2008 the curtain was closed temporarily on its 50-year history. It was torn down to be rebuilt as a new hall.

Nakanoshima Festival Tower (Fig. 1) was planned as a 200-meter high skyscraper complex comprising commercial facilities and offices as well as the rebuilt Festival Hall. Based on this plan, the new hall was reborn as a cutting-edge hall that inherits the tradition and acoustics of the old hall and provides improved functionality and amenities for performers and the audience. In addition, it was planned with the purpose of contributing to the promotion of urban revitalization by enhancing the cultural functions and central business functions on Nakanoshima by including commercial facilities and offices in the building.

The building owner is The Asahi Shimbun Company. Design and supervision was performed by Nikken Sekkei Ltd. Construction was performed by Takenaka Corporation. The construction period was 34 months from January 2010 to end of October 2012.

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2. Architectural Overview

Fig. 2 shows a cross section of the building. The building is comprised of 3 broad sections: the lower-level floors from the 8th floor and below including the hall; the intermediate-level floors from the 9th floor directly above the hall to the 12th floor; and the upper-level floors from the 13th floor which includes the Sky Lobby.

The new hall (Fig. 3) occupies floors 3 through 8 of the lower-level floors. It is a large-scale hall with the same maximum seating capacity of 2,700 seats as the old hall. The stage space has been greatly expanded and has specifications to meet the demands of the latest performing arts, such as a high and deep fly tower.

The first and second floors under the hall have entran-

Building name:
Nakanoshima Festival Tower
Building owner:
The Asahi Shimbun Company
Design/Supervision:
Nikken Sekkei Ltd.
Construction:
Takenaka Corporation
Building address:
2-22, Nakanoshima, Kita-ku, Osaka
Application:
Office building, theater, stores, etc.
Site area:
Approx. 8,200 m^2
Building footprint:
Approx. 5,700 m^2
Total floor space:
Approx. 146,000 m^2
Number of floors:
39 above-ground floors;
3 underground floors
Height:
Approx. 200m
Foundation depth:
16.5m below ground level
Construction period:
January 2010 to end of October 2012

Figure 1. Photograph of entire scene.

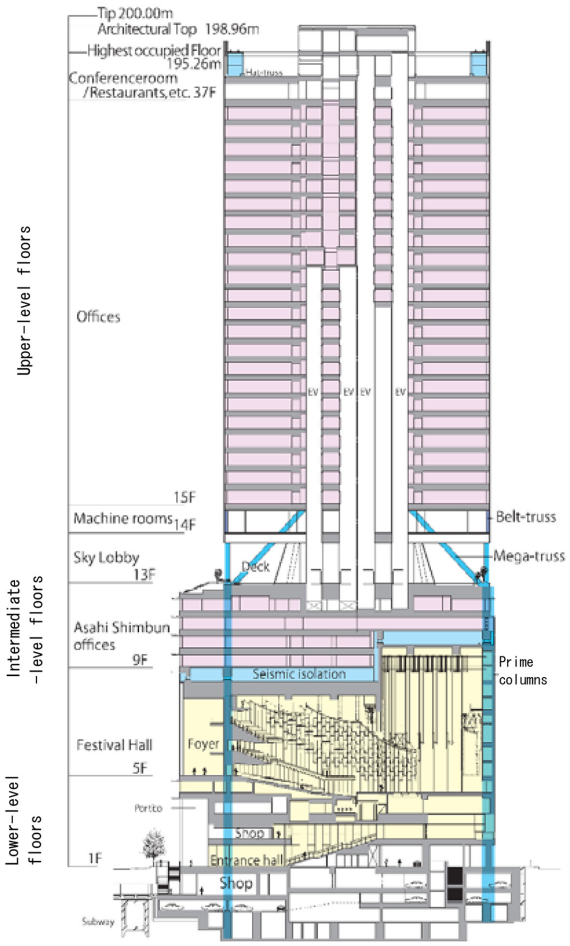


Figure 2. Cross section of the building.

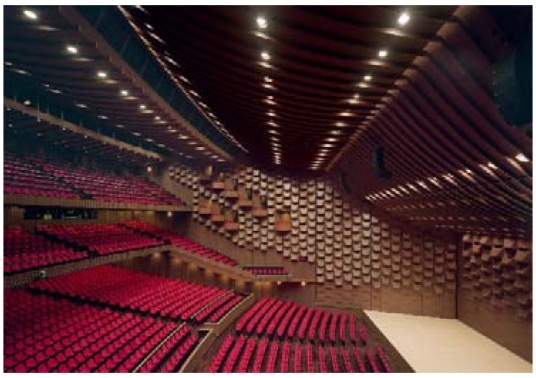


Figure 3. Interior of the new Festival Hall.

ces to each of the various facilities including the hall together with commercial facilities and offices. The below-ground floors contain commercial facilities that connect with the underground shopping mall along with parking garages, machine rooms, etc.

The application of the intermediate-level floors is offices, and Asahi Shimbun Co. uses them as its headquarters.

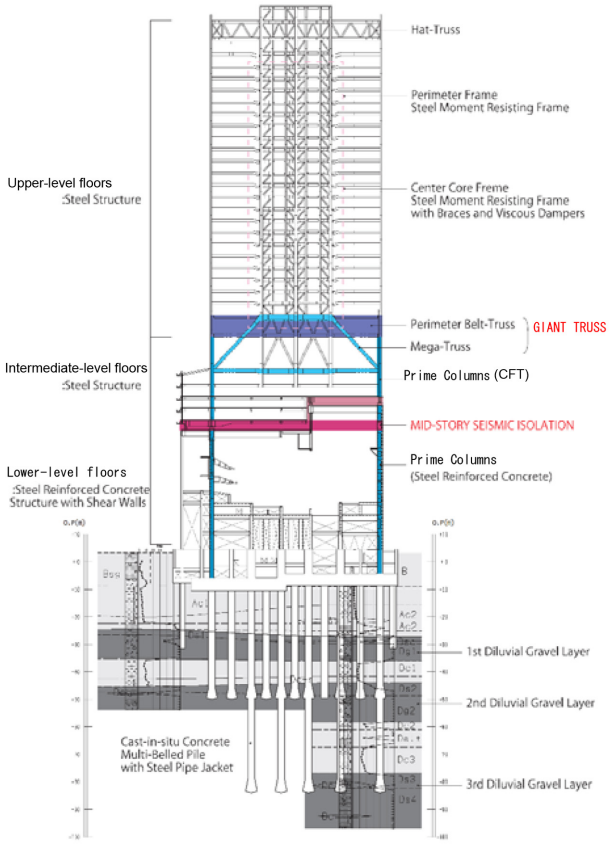


Figure 4. Cross section of building framework.

Above that, on the 13th floor is the Sky Lobby, a shared lobby serves as the transfer floor to elevators providing access to upper-level floors. The upper-level floors are used mainly for tenant offices. The offices in the upper-level floors are laid out in a center core format for high-efficiency rental office usage, providing an office space without columns for approximately 15 m from the outer walls.

3. Structural Outline

The most important proposition for this building from a structural planning aspect was how to achieve the building up of center-core high-rise offices above the large 2,700-seat capacity hall while maintaining high structural performance. In order to implement this proposition, the following two points that characterize the structural plan of this building were employed:

Giant trusses to transfer the load of upper-level floors to the perimeter of the hall and secure the large open space of the hall.

Mid-story seismic isolation system to create a seismic isolation layer in the boundary between the hall and the office floors.

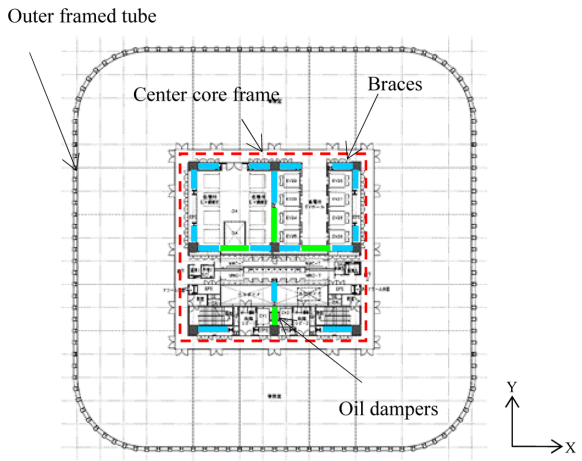


Figure 5. Basic floor layout of upper-level floors.

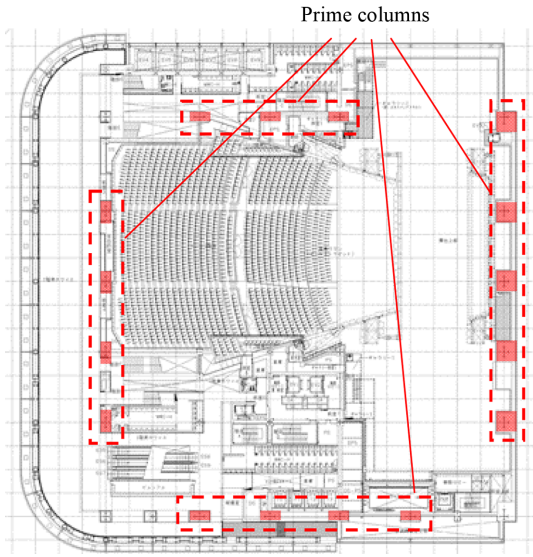


Figure 6. Basic floor layout of lower-level floors.

Fig. 4 shows a cross section of the building framework. The structure type is steel-reinforced concrete (SRC) construction for the lower-level floors including the hall. Steel frame (S) construction is applied for the intermediate-level and upper-level floors used as offices. A mid-story seismic isolation system is employed as a seismic isolation layer directly above the hall between the lower-level floors and the intermediate-level floors.

The upper-level floors have a center-core floor layout (Fig. 5). As a structural plan, in addition to the core frame with braces (center core frame), they also have 128 H-shaped steel columns at 1.8 m intervals forming a bearing-wall-like perimeter framework (outer-framed tube) to ensure the required strength and stiffness. CFT (concrete-filled tube) is employed for the 9 columns of the core, and oil dampers are installed as wind sway measures and for response suppression during earthquakes. In addition, a hat truss is installed in the top of the building to suppress warping of the building as a whole.

The 13th and 14th floors between the intermediate-level and upper-level floors has giant trusses which is one of the major characteristic features of this building. In this section, two major types of trusses, - mega-trusses and belt trusses -, are installed. The other characteristic feature of this building, the mid-story seismic isolation layer, is installed between the lower-level floors and the intermediate-level floors.

For the lower-level floors below the seismic isolation layer (Fig. 6), SRC construction was used in considera-

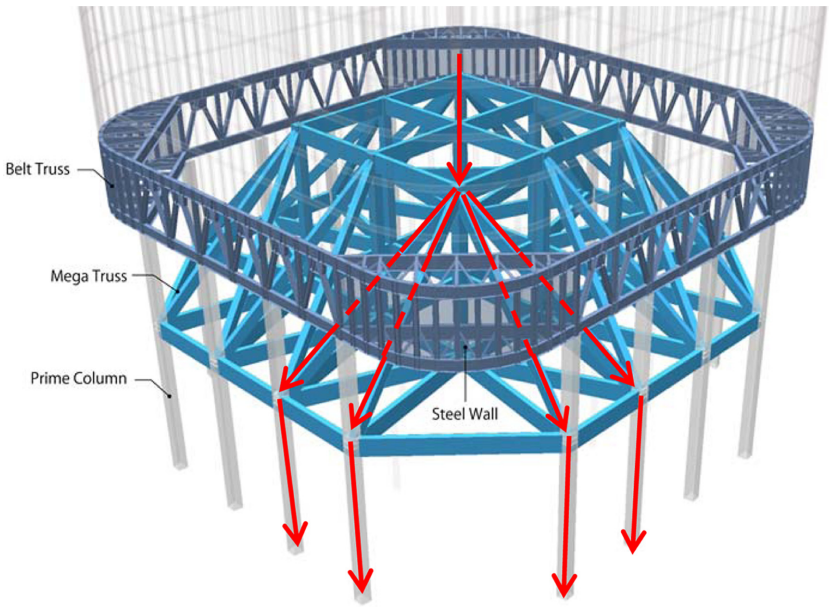


Figure 7. Outline of mega-trusses and belt trusses.

tion of the sound isolation properties for the hall, and was planned with shear walls of sufficient volume to resist seismic forces. By the use of the giant trusses, the entire weight of the upper-level floors was transferred to the 16 columns stand outside of the hall. The columns, which are referred among the people involved as "prime columns", become giant SRC columns with a cross section of 3.0 m × 1.5 m in the lower-level floor section.

The foundation is cast-in-place concrete piles with steel casings on the upper sections, and the support layer is the third diluvial gravel layer located at approximately 86m under the ground level at its deepest place. In order to ensure that the piles have the high support strength required, multi-step enlarged diameter piles, in which the diameter is enlarged in the intermediate section of the pile in addition to the enlarged section at the pile tip, were employed.

4. Mega-Trusses / Belt-Trusses / Prime Columns

The mega-trusses are gigantic three-dimensional truss structures with a height of approximately 20 m extending from the floor of the 13th floor to the floor of the 15th floor. They support the total load of approximately 38,000 t borne by the 9 CFT columns of the core of the upper-level floors, and that load flows to the 16 large-cross-section columns, the prime columns, which emerge directly below the outer perimeter section of the upper-level floors.

On the other hand, the belt trusses are planar trusses installed as strips around the perimeter of the 14th floor, and perform the work of consolidating the axial forces of the 128 columns standing around the perimeter of the upper-level floors into the prime columns. As a result, the prime columns become the columns that support the entire load of the 13th floor and above, and by causing all of the load for the upper-level floors to flow via the mega-trusses and belt trusses to the prime columns, the large hall space of the lower-level floors was realized.

The framing plan and framing elevations for the mega-trusses and prime columns are shown in Figs. 8 and 9. The mega-trusses are comprised of the 13th floor girders (Fig. 9) which are the lower chord members, the girders inside the core on the 15th floor (inside the core in Fig. 8) which are the upper chord members, and the diagonals which connect them with a distance of approximately 27 m between nodes (outside the core in Fig. 8). The long-term load supported by each diagonal is approximately 3,000 t.

The prime columns are columns with large cross sections which emerge directly below the outer perimeter of the upper-level floors, 4 columns each in the east, west, north, and south for a total of 16 columns, and support the entire weight of the upper-level floors and transmits that load through the mid-story seismic isolation layer and lower-level floors to the piles. For the intermediate floor levels for which they compete with the mega-trusses, they support a long-term load of approximately 6,000 t. In this

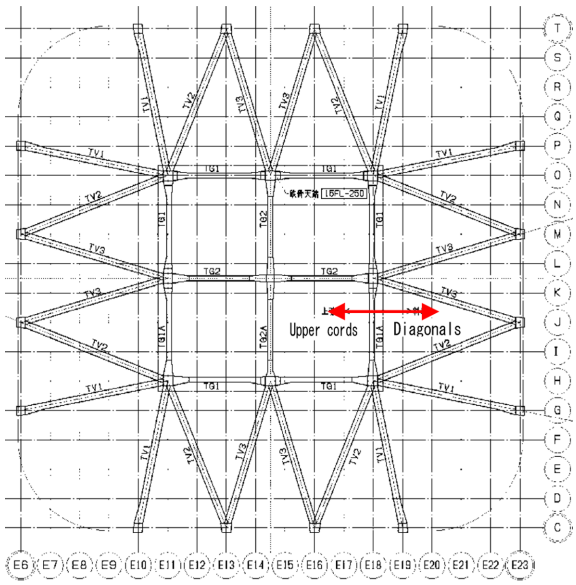


Figure 8. 15th floor framing plan.

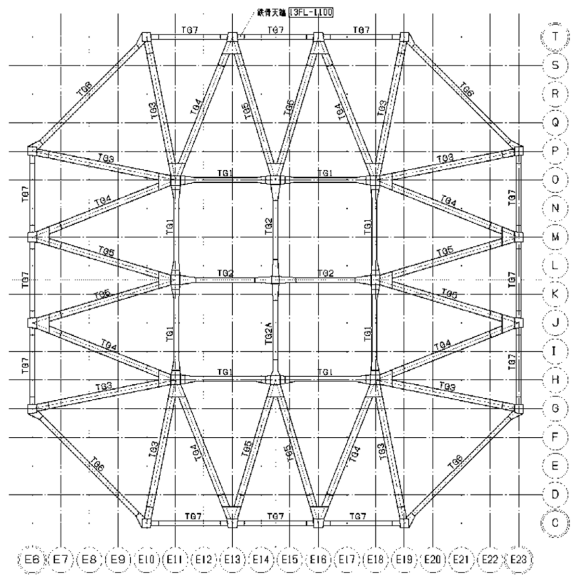


Figure 9. 13th floor framing plan.

mega-truss and prime column design, design and investigation was performed with particular focus on strong diagonals that could reliably support axial forces of several thousand tons, reducing thrust deformation to the maximum extent possible, and details that could reliably transfer stresses.

The mega-truss diagonals are box braces with a parallelogram cross section. That part of the cross section list is shown in Fig. 10. Since they are extremely important members supporting the upper-level floors and are compression members under high axial force, SA440B (440 N/mm^2 yield strength and 590 N/mm^2 tensile strength)

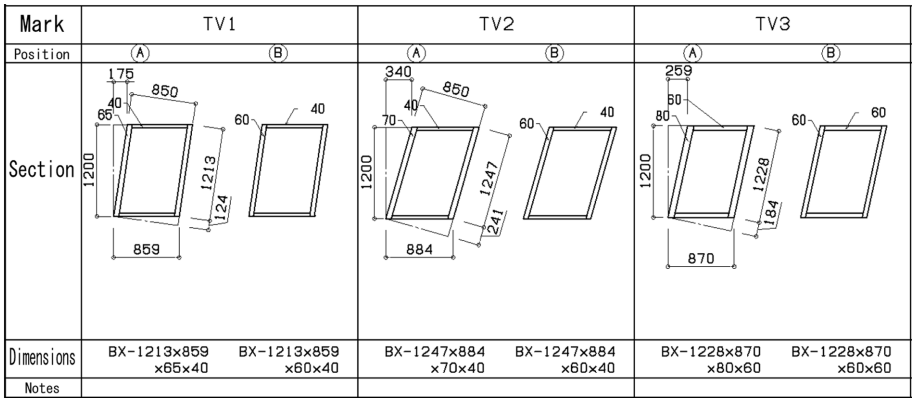


Figure 10. Cross section of mega-truss diagonals.

with a plate thickness of 40 to 60 mm (with a maximum of 80 mm in the joint sections) is used as the steel material. In order to reduce the buckling length, they are subject to horizontal deformation restrictions from the 14th floor surface at the locations where they penetrate the 14th floor.

Suppression of thrust deformation was handled by securing the maximum member cross-sectional area for the upper chord and lower chord members. By suppressing the long-term axial stresses of both members to an average of 70 N/mm^2, thrust deformation in each direction was suppressed to around 10 mm or less. Since the degree of stress is small, JIS SN490B (325 N/mm^2 yield strength and 490 N/mm^2 tensile strength) was used as the steel material, but it has become an extremely thick large-cross-section box girder with girder formation of 1,500 to 2,000 mm and a maximum plate thickness of 90 mm.

The prime columns are lined up as 4 each on the east, west, north, and south sides, with the inside columns each having 2 diagonals and 2 lower chord members with a trapezoid cross section (left side of Fig. 11). The outer 2 columns each have 1 diagonal and 1 lower chord member with a parallelogram cross section (right side of Fig. 11). SA440C was used as the steel material. Fc90 (90 N/mm^2 cylinder strength) was used as the fill concrete. The plate thickness of the steel material was a maximum of 100 mm in the joint sections.

The various members of the mega-trusses and the prime columns contend three-dimensionally (Fig. 12), and a maximum of 8 members converge on a single node. Because of this, for the design of the mega-trusses, the longest time was spent on decisions on a joint shape which is rational and manufacturable. In the initial stages of the design, we considered the use of circular steel pipe for the diagonals and cast steel for the joint sections. However, manufacturing within the range of transportable and liftable weight would be difficult. Therefore, design proceeded with the current proposal of creating joint blocks with a welded structure of extra-thick steel plate where multiple members converge.

During design, particular attention was paid to the continuity of the plate material for the smooth transmittance of the stresses of each plate comprising the members. The reason that the diagonal cross section is a parallelogram box cross section is to have a resulting shape in which the upper and lower surfaces of adjacent diagonals would be in the same plane, and in addition, the vertical surfaces on both sides would compete with the skin plates of the box columns. The trapezoid shape or parallelogram shape of the prime columns was also decided through considera-

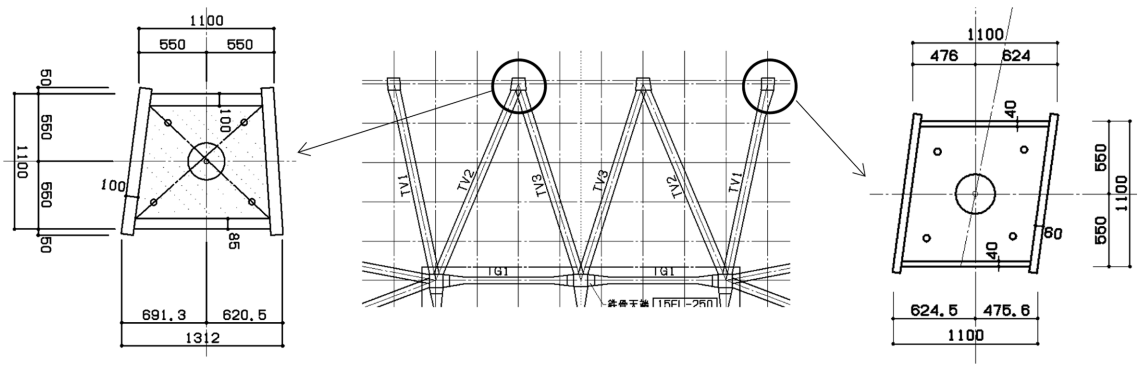


Figure 11. Cross section of prime columns.

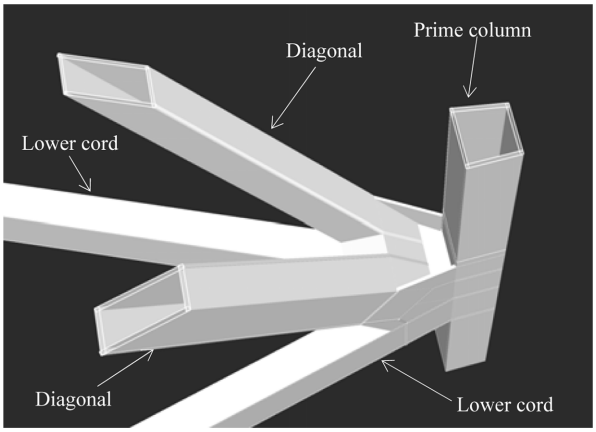


Figure 12. Schematic diagram of Mega-truss connection.

tion of the continuity of the plate materials of the diagonals and lower chords. By employing this cross-section shape, simplification of the joint section and smooth transmittance of stresses was achieved.

Higher safety criteria during earthquakes were set for the various members which comprise the mega-trusses than for other members. For horizontal forces, design was performed so that even when the design seismic load set based on the results of dynamic response analysis for the occurrence of an "extremely rare" strong earthquake was set to 1.5 times as large. The degree of stress that occurs in members is within the short-term allowable stress. Although a maximum plate thickness of up to 100 mm is used in the vicinity of the prime column joints, the localized stress of each joint component has been verified by FEM analysis (Fig. 13). We worked to secure sufficient safety since they are extremely important structural members supporting the building.

5. Mid-Story Seismic Isolation

The other characteristic feature of this building, the mid-story seismic isolation layer, is installed between the lower-level floors and the intermediate-level floors. Fig. 14 shows the layout of the seismic isolation layer. Rubber bearings with lead plugs (LRB: Lead rubber bearings) are employed as the isolation devices. Oil dampers are employed as energy-absorbing elements. Two extra-large 1,500 × 1,500 mm square LRBs (Fig. 15) in a set are used directly under the prime columns in order to support the prime column axial forces of as much as 6,000 t. 16 sets of the large square LRBs are set to support the prime

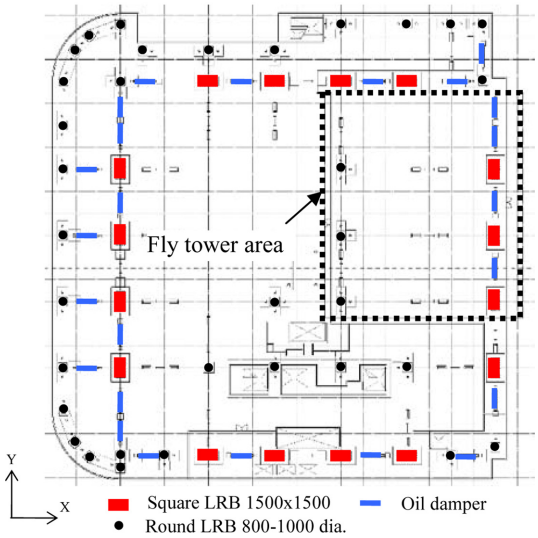
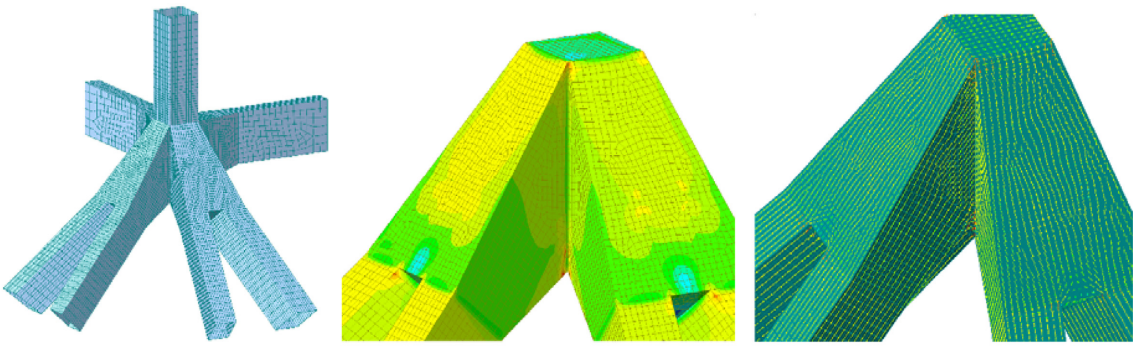


Figure 14. Schematic layout of seismic isolation floor.

Figure 15. Square LRB under deformation capacity test.



FEM model | Effective stress distributions | Principle stress directions

Figure 13. FEM analysis model and response status.

Table 1. Seismic-resistant design criteria

Earthquake scale			Level 1 earthquake	Level 2 earthquake
Target performance	Upper-level floors Intermediate-level floors	Member stress	Short-term allowable stress or less	Short-term allowable stress or less
		Story drift angle	1/300 or less	1/150 or less
	Seismic isolation layer	Deformation level	1/2 stable deformation or less	Stable deformation or less
		Deformation amount	200mm or less	400mm or less
	Lower-level floors	Member stress	Short-term allowable stress or less	Short-term allowable stress or less
		Story drift angle	1/800 or less	1/400 or less
	Foundation	Member stress	Short-term allowable stress or less	Short-term allowable stress or less
		Support strength	Short-term allowable bearing capacity or less	Short-term allowable bearing capacity or less

columns which bear 95% of the building weight above. The remaining 5% weight is supported by 800~1000 mm diameter round LRBs. Oil dampers of maximum resistance of 1000 kN were installed as energy absorption devices along with each orthogonal directions X and Y. The total number of the oil dampers is 24 (12 for each direction). In order to avoid crashing, the clearance between the object isolated side and the object fixed side is secured as 650 mm. In addition, since the seismic isolation layer of this building is installed directly above the hall, the area of the upper section of the stage called the fly tower becomes a staggered seismic isolation layer which is 2 stories higher and the clearance between the structural components of isolated side and the fly tower are secured as 750 mm at least.

By employing this mid-story seismic isolation structure, this building is able to achieve the high earthquake-resistance performance required of the headquarters of a news company and maintain primary building functions when a large earthquake occurs. This building's seismic-resistant design criteria are shown in Table 1. Since the building employs a mid-story seismic isolation structure, the building was divided into the section above the seismic isolation layer (upper-level floors and intermediate-level floors), seismic isolation layer, and the section below the seismic isolation layer (lower-level floors). Criteria was set for each layer.

The seismic performance of the mid-story seismic isolation system is verified through series of dynamic response analyses using two levels of variation of earthquakes set created in accordance with Japanese law. The intensity of the motion is set as “rare” earthquake for Level 1 and “extremely rare” earthquake for Level 2. Dynamic response analyses was performed using six to ten varieties of earthquakes including recorded motion data and artificially generated motion considering geological properties of the specific site.

For the upper-level floors and intermediate-level floors in the section above the seismic isolation layer, member stress was set at the short-term allowable stress or lower even for the occurrence of a Level 2 earthquake. The maximum story drift angle was set at 1/150 or less. The lower-level floors and the foundation are not a section where seismic isolation effect is exhibited. It becomes a typical earthquake-resistant structure with RC shear walls. In view of its importance as the structure supporting the upper-level floors and intermediate-level floors, member stress for the occurrence of a Level 2 earthquake was set at the short-term allowable stress, or lower. As the seismic resistance criteria of the seismic isolation layer, the deformation amount for the occurrence of a Level 2 earthquake was set at 400 mm or less. This corresponds to the stable limit deformation amount of the 800 mm minimum diameter of the LRBs used.

Figs. 16 and 17 show examples of results of dynamic response investigation. Since the lower-level floors are SRC construction as rather rigid structure, seismic force is largely amplified at the top of the hall. Mid-story seismic isolation layer reduces the acceleration into the inter-

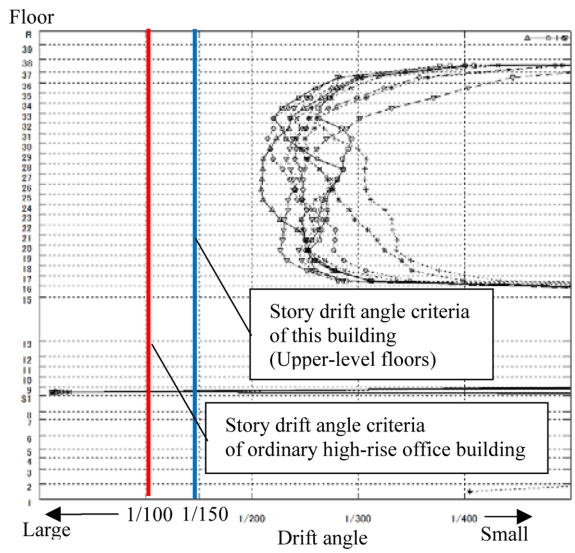


Figure 16. Example of story drift angle responses (expression is inverted; large drift goes to the left).

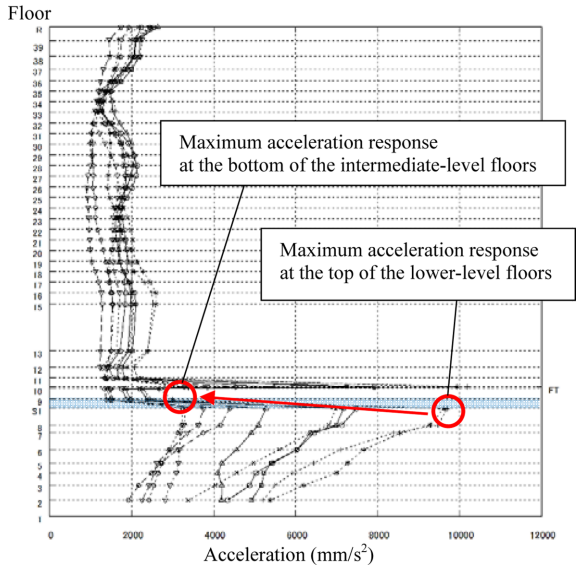


Figure 17. Example of floor acceleration responses.

mediate-level floors to around 25%. The maximum story drift of the upper level floors is more than 30% smaller than that of general high-rise office buildings. It was verified through dynamic response analysis that even facing very strong earthquakes which are determined by Japanese law as "extremely rare", the stress occurs in the structural members of this building does not exceed short-term allowable stress of the materials. By employing this mid-story seismic isolation structure, the building is able to achieve the high seismic safety required of the headquarters of a news company.

Due to mechanical properties of rubber bearings, tension forces in a rubber bearing as a result of overturning moment and vertical seismic force shall be suppressed in design of seismic isolation system. This building has the most efficient structural system to avoid the rubber bearings subjected to tensile force because of application of mega-trusses. Since the building weight is concentrated to the large square LRBs which are laid out along perimeter line of the upper-level floors, the uplift force caused by overturning moment is theoretically minimized. It was verified that the large square LRBs are always under compression stress during extremely rare strong earthquakes even considering coupling effect of overturning caused by horizontal earthquake and uplift effect caused by vertical earthquake.

6. Construction of Mega-Trusses

6.1. Cooperative system

This building has special steel frameworks that exceed the common architectural steel frame categories. For the fabrication and construction, a working group was created comprising designers/supervisors (Nikken Sekkei), construction workers (Takenaka Corp.), and personnel related

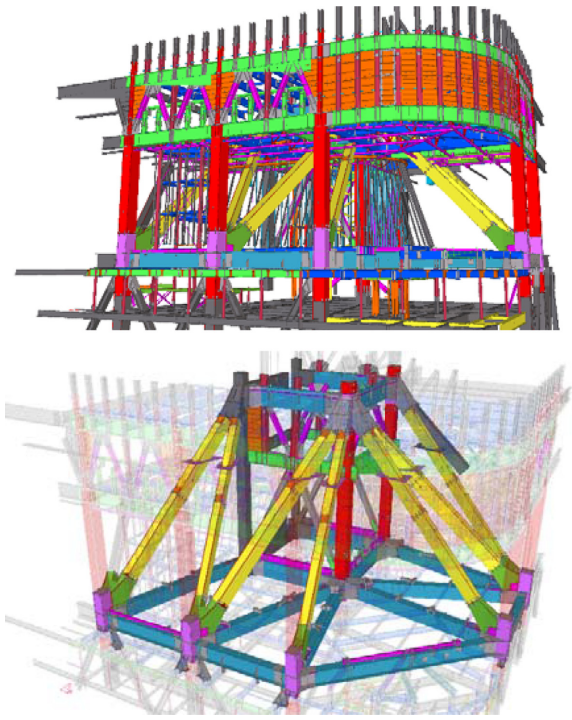


Figure 18. Example of 3D CAD modeling.

Figure 19. Full scale model of mega-truss connection.

to the steel framework fabrication companies (welding specialists from the 4 fabrication companies and third-party inspection companies, etc.), and through performing technical consultation spanning approximately 1 year from the start of construction, problem resolution to ensure construction quality was achieved. Even after starting the actual fabrication, the cooperation network beyond company continued and they worked to improve quality.

6.2. Factory fabrication of mega-trusses

Fabrication of mega-trusses was the theme that occupied the most time for discussions in the working group. Because they are complex shapes in which members with irregular-shaped box cross sections come together in three dimensions, in the extraction of issues regarding shape

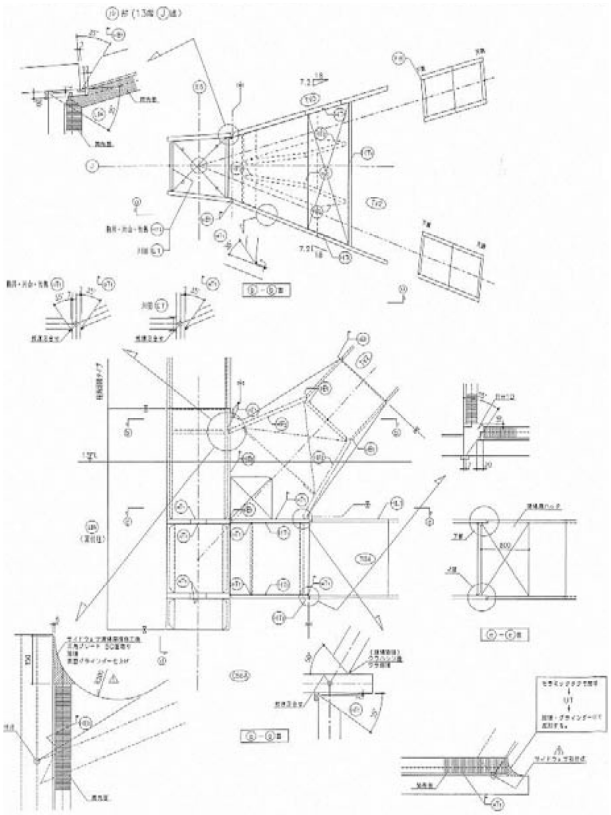


Figure 20. Example of welding scheme diagram.

confirmation and fabrication in the working group, the 3D CAD used for steel framework diagram generation (Fig. 18) and full-scale models (Fig. 19) were employed.

Fig. 20 shows an example of the welding scheme diagram for the joint section. For the mega-trusses, optional welding scheme diagrams were prepared for the various locations, and the working group checked everything from the welding sequence to groove shapes and methods for penetrating backing metals. Even in the actual fabrication, each company created their own special jigs or other methods for turning and positioning the members, and welding was performed with flat welding that would secure high-quality welding.

Furthermore, for the joint sections where diagonal members came together with columns, etc., since there were three-dimensional influences such as the influence that errors in planar angles also had in the height direction, detailed dimensional accuracy control was important. For this point as well, careful consultations on the control policy for relative angle dimensional accuracy of members were conducted in advance (Fig. 21). Planning for the assembly order for each member component while carefully predicting the influence of shrinkage or deformation due to welding (Fig. 22) was performed. During actual fabrication, the steel framework fabrication companies worked to secure fabrication accuracy by incorporating their original ideas in their processes, such as using deformation-restricting materials, introducing contrary warping, performing three-dimensional measurement and pro-

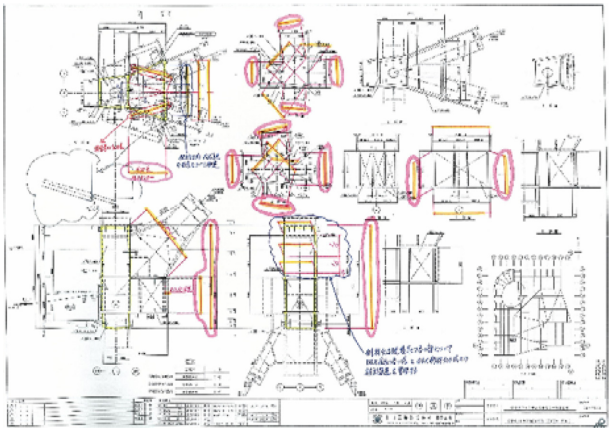


Figure 21. Record of dimensional accuracy control policy discussion.

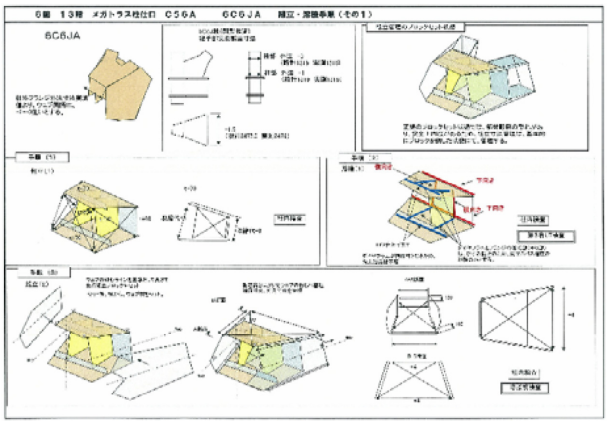


Figure 22. Example of assembly order diagram.

Figure 23. Photographs of fabrication status of mega-trusses.

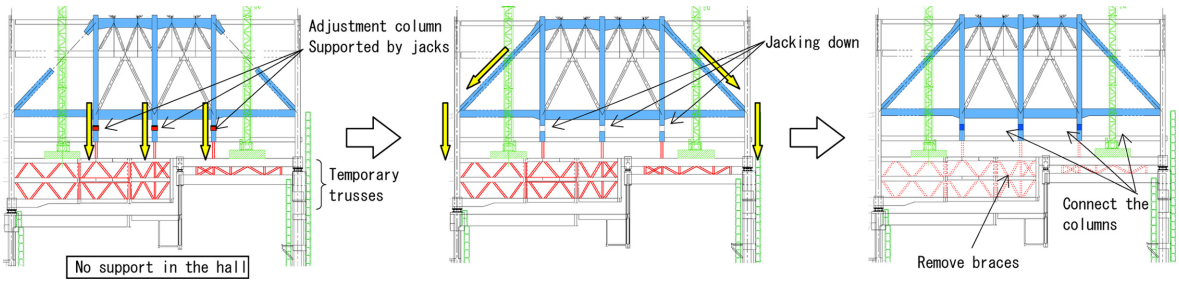


Figure 24. On-site construction procedure for mega-trusses.

viding feedback after each process, etc.

The factory fabrication of mega-truss steel frameworks required a period of about 4 months. Fig. 23 shows the fabrication status at a factory. In particular, for a member with a weight of approximately 20t per piece where it was necessary to perform CO_2 arc welding for almost all of the welding, the completion of a single part required approximately 2 months.

6.3. On-site mega-truss construction

Fig. 24 shows the on-site construction procedure for mega-trusses. One of the issues in erecting a mega-truss is how to support the up to 4,500 t weight of the mega-truss steel frame being erected without halting hall construction in the lower-level floors. For this construction, a method in which a temporary truss was installed in the intermediate 9th to 11th floors to transfer the load during erection of the mega-truss to the perimeter prime columns was employed. The 12th floor core column connecting what should be called a temporary mega-truss with the actual mega-truss being erected was used as the adjustment column, that was divided vertically and jacks were placed in between to enable height adjustment to be performed while supporting the load. After the erection of the mega-truss, jacking down was performed to transfer the load to the final load-transmittance system.

Regarding the member division of the mega-truss, the pieces were specified as being under around 25 t considering transport restrictions and tower crane capacity. However, since as the number of pieces increases the amount of on-site welding work required also increases, in order to reduce on-site welding, an off-site yard was secured at a separate site in the Nakanoshima District so that transportation cold be performed without having to cross bridges with strict weight restrictions (Nakanoshima is a sandbank surrounded by two rivers). Some girder members were connected at that site and then transported to the actual construction site. In the cases of lifting members with a maximum weight of 39 t, lifting was performed by tandem lifting using two tower cranes. For the erection, three-dimensional measurements were performed to control accuracy, and for the installation of the last

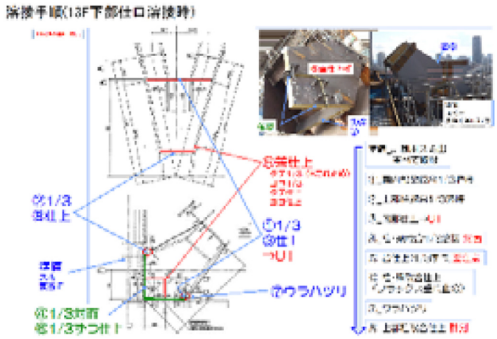


Figure 25. On-site welding procedure instruction.

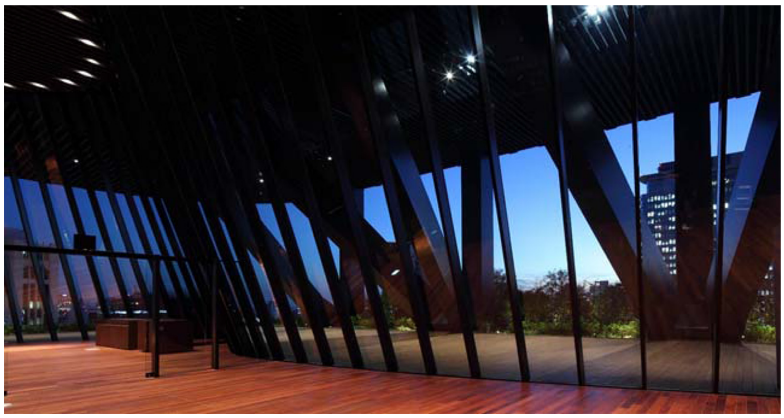


Figure 26. View of Mega-truss diagonals from Sky Lobby.

member of the diagonal members, final processing dimensions were determined through on-site measurements.

For the on-site welding construction work, since repairs of flaws in the extra-thick materials would not be easy, on-site welding work was performed according to procedures had a particular emphasis on securing welding quality. In order to enable bottom flanges of box girders to be flat-welded, some part of the top flanges were opened using post-construction processes, etc. Also from the viewpoint of securing accuracy, on-site welding of the joints or fittings of large-section box members was performed by two welders working face to face, and the detailed welding procedures for each part were decided in advance (Fig. 25).

The on-site construction period was 4 months and with the performance of 240,000 m of on-site welding when expressed in terms of 6 mm, jacking down was completed on May 11, 2011 and the mega-truss was completed.

7. Conclusions

Since the start of service, the Nakanoshima Festival Tower gave off a feeling of presence and elegance in the Nakanoshima neighborhood. The building welcomes all visitors into the Sky Lobby and everyone can see closely and touch the steelwork of the mega-truss diagonals and the prime columns. This steelwork now become not only an important structural element but also a key part of architectural elegance. All engineers who worked on the construction of the mega-truss are happy and proud of it.