DESIGN AND CONSTRUCTION OF ROOFTOP GREENING CANOPY OVER WHOLE BUILDING

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Abstract

In earthquake-ridden Japan, rooftop structures as well as the main structures of buildings are required to be highly seismic resistant. In the New Miyashita Park Project that we will present, we developed the whole rooftop area of the three-story commercial facilities into an approximately 8,700 square meter city park and designed a canopy made of steel for greening to cover the entire park (Fig. 1). Seismic force, greening system and the beauty of the structure were the key challenges in designing the ROOFTOP GREENING CANOPY. We would like to present (1) our concept of how to calculate the seismic force in designing a large-scale rooftop structure and (2) the structural design of a simple and minimal structural frame with a greening system and earthquake resistance.

We needed to properly determine the design seismic force in designing a large-scale rooftop structure. Therefore the horizontal seismic coefficient in consideration of the response of the main structure below was calculated to determine the design seismic force. The horizontal seismic coefficient was calculated by the eigenvalue analysis considering the weight ratio and stiffness ratio of the rooftop structure to the structure below. Thus the horizontal seismic coefficient for this project was set to 0.86 G.

The overall structural planning is based on a simple frame made of two oval arches connected with four beams. This structure can resist a large seismic force on the rooftop structure with a small number of members. The structure has two major features: One of them is a continuous series of independent frames in which the members are arranged in an A-shape in the ridge direction. The other feature is a reduction of the rise in the span direction, which has lowered the center of gravity position and consequently contributed to ensuring of the horizontal stiffness and to reduction of stress caused by an earthquake.

The principal framing structure of the canopy covering the whole site consists of circular steel tubes (356.6 mm in diameter), in the repeated elliptical shapes of different sizes ranging between 9.0 and 16.0 meters in height and between 25.0 and 35.6 meters in span, which are supported mainly by the beams at the rooftop and third floor levels of the main structure.

Keywords: rooftop structure; canopy; eigenvalue analysis; seismic design

1. Introduction

Rooftop structures of buildings, as well as the main structures, are required to be highly seismic resistant in Japan. This report presents a case of seismic design of a large-scale structure located above a building in Japan.

2. Overview of Project

This is a project for rebuilding the existing Miyashita Park which was situated above the parking space constructed in 1964. For the new park, we had to keep a green space equal to that of the existing park. However, if such a green space is accommodated in the rooftop park area, that will reduce the space available for the park's activities. Under the circumstances, this project is aimed at securing a green space above the park by planning a canopy for greening on the roof of new three-story commercial facilities, as well as securing a space for the park's activities.

For the lower-floor commercial facilities, the maximum rentable floor space needs to be secured because of the high-rent urban commercial property, which causes the exterior walls of the building to align with the site configuration. Likewise, the canopy over its top was also required to take a shape in alignment with the site configuration. Thus this project will realize a green canopy all over the 330-meter-long park (approximately 8,700 square meters in area), split into the northern and southern parts, which is located 18 meters above the ground.

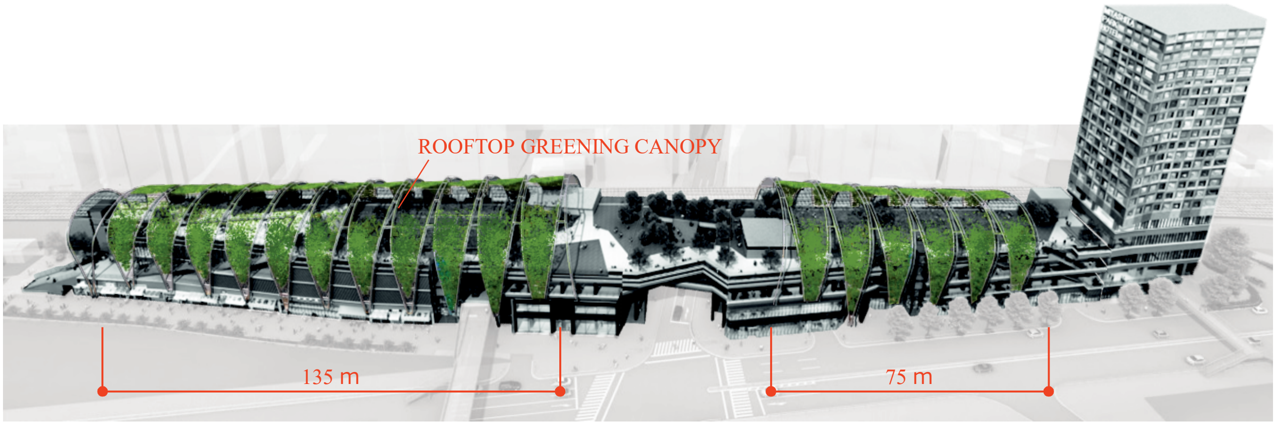


Fig. 1 – Rendering of whole project

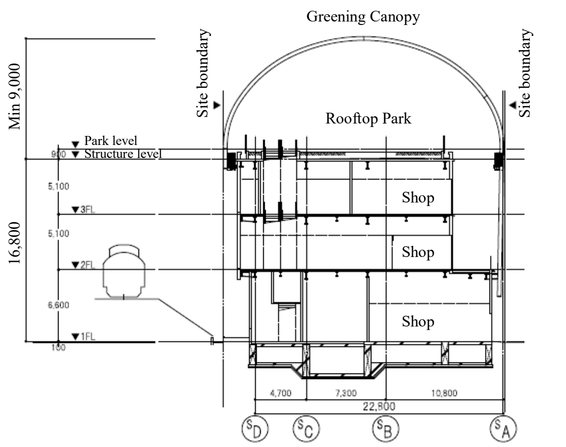


Fig. 2 – Sectional view (Short span)

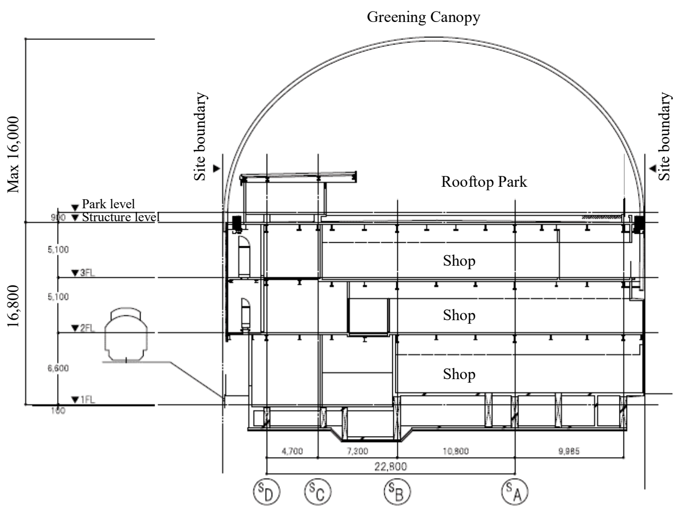


Fig. 3 – Sectional view (Long span)

In order to achieve a three-dimensional shape in alignment with the site configuration, we focused on continuous arch shapes that match a long span. If these arch shapes are firmly connected to each other, that will create a need to ensure transmission of horizontal forces under horizontal loading and the surface stiffness because the stiffness varies from span to span, which will increase the difficulties in designing and fabrication.

Hence, we arranged this shape in a continuation of a unit shape with two arches combined in an "A" shape. And then the units were softly connected to each other with a wire, which has contributed to enhancement of the degrees of freedom in the span and layout, resulting in a configuration which can be easily located according to the site configuration. Furthermore, this shape enables each unit to be free-standing and consequently meets the conditions of construction on a narrow and long site.

We also installed meshes for climbing plants on the stainless steel wires that connect the units to each other, thus achieving the openness of a park and an interior space covered with foliage.

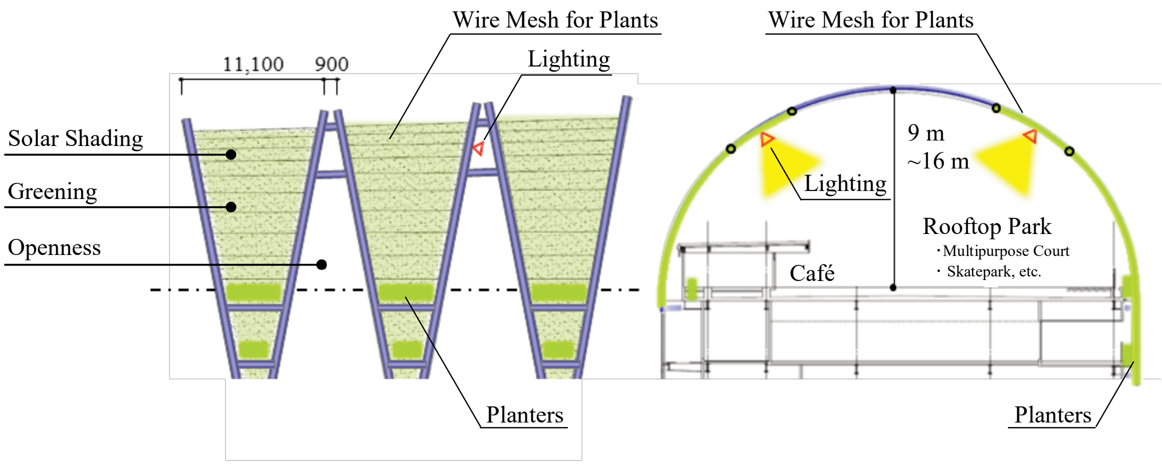


Fig. 6 – Greening and Lighting System

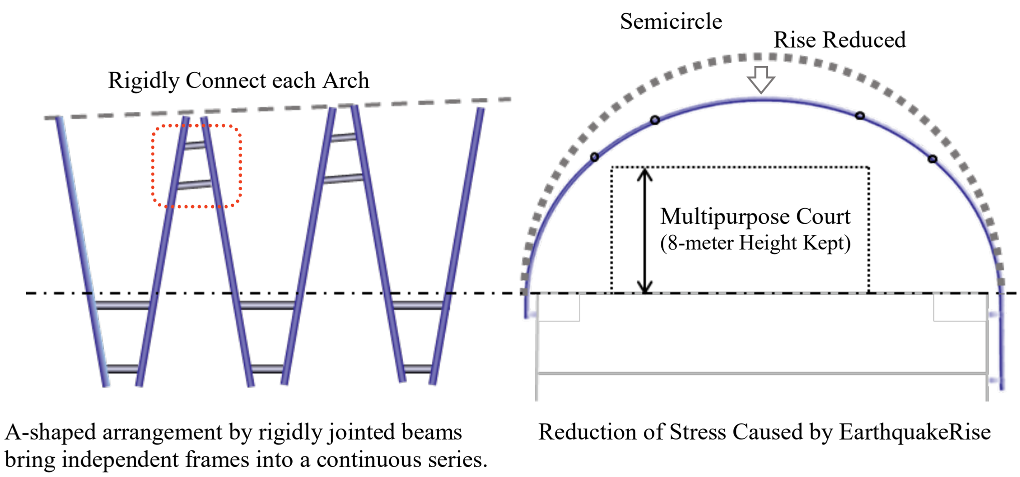


Fig. 7 – Improvement of earthquake resistance

3. Definition of Canopy Configuration

The entire configuration of the canopy needed to be continuously varied as a shape that covers the whole site smoothly, which required a new rule for laying structural members out.

Therefore, a 3D curved surfaces was defined from the elliptical shapes based on the site, structural height and greening conditions in this project, and a smooth configuration in alignment with the site configuration was achieved by locating structural frame members on the cut surfaces of the 3D configuration.

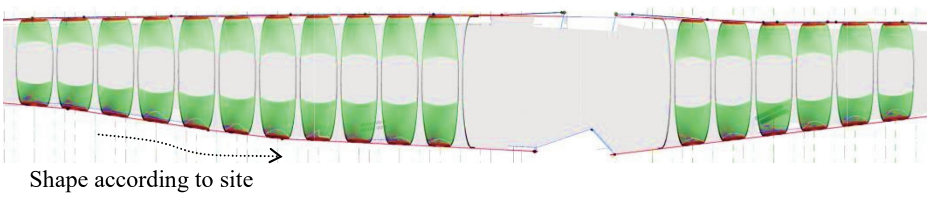


Fig. 8 – Greening layout (plan view)

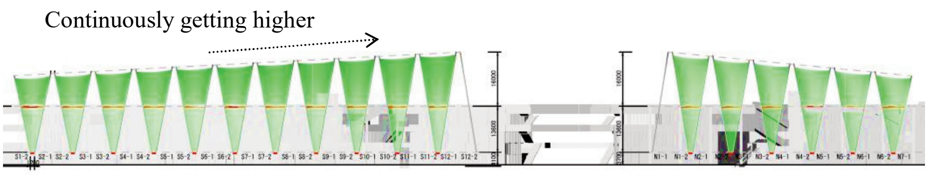


Fig. 9 – Greening layout (elevation)

(1) The greening layout was planned by forming 3D curved surfaces based on the elliptical shapes under the site boundary conditions and the width and height constraints.

(2) The 3D curved surfaces were cut to match the elevational shape, which enabled us to successfully plan a façade design without being restricted by the site configuration.

(3) A-shaped structural frame members were placed at the cut positions of the 3D curved surfaces.

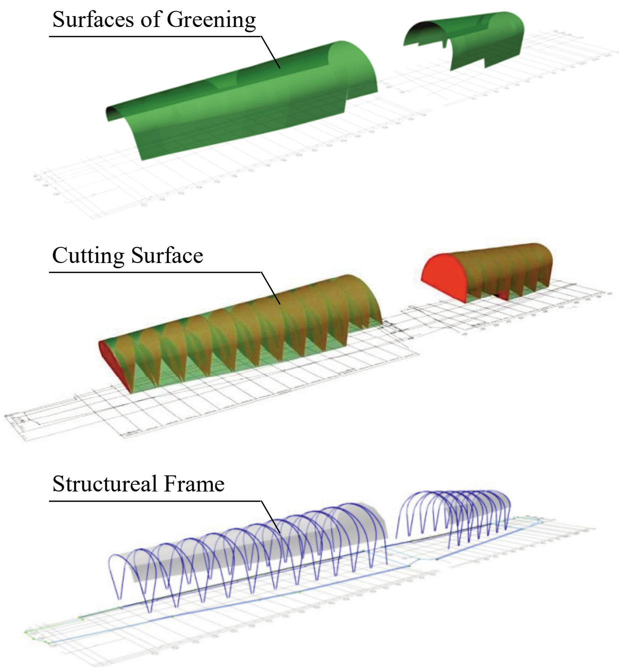


Fig. 10 – Rule for laying structural members out

We placed the structural members, following the above rule, and as a result, were able to replace the continuously varying 3D curved surfaces by a continuity of simple A-shaped frames.

4. Establishment of Design Seismic Force

We needed to properly determine the design seismic force in designing a large-scale rooftop structure. To establish the design seismic force for the rooftop canopy, the response of the structure below had to be considered because the canopy is installed on the top of the three-story building. It was anticipated that the response of the structure above would increase if we consider the stiffness ratio and weight ratio of the rooftop structure to the building below.

The horizontal forces that will be applied to the rooftop structure in an earthquake can be determined by two methods: One method is by means of seismic response analysis, and the other by eigenvalue analysis and vertical distribution coefficient of Japan Code. In the project discussed in this paper, we determined the horizontal force that would be evaluated on the conservative side against the rooftop structure by applying an analytical method for showing larger horizontal force from the results of the above two types of analyses.

Thus in designing the canopy for this project, the horizontal seismic coefficient was calculated by the eigenvalue analysis considering the weight ratio and stiffness ratios of the rooftop structure to the structure below and consequently set to 0.86 G. The eigenvalue analysis applied for this project is as follows:

The target horizontal deformation of the canopy steel structure in an earthquake was set to 1/100, and the horizontal seismic coefficient was found through eigenvalue analysis. The horizontal stiffness was corrected according to the result. Then convergence calculation was performed to find the horizontal seismic coefficient.

First, the horizontal seismic coefficient was calculated on the basis of the assumption that the deformation was 1/100 at the horizontal stiffness of the canopy equal to 0.5 G in the typical range shown in Fig. 11 for establishment of the horizontal seismic coefficient, where:

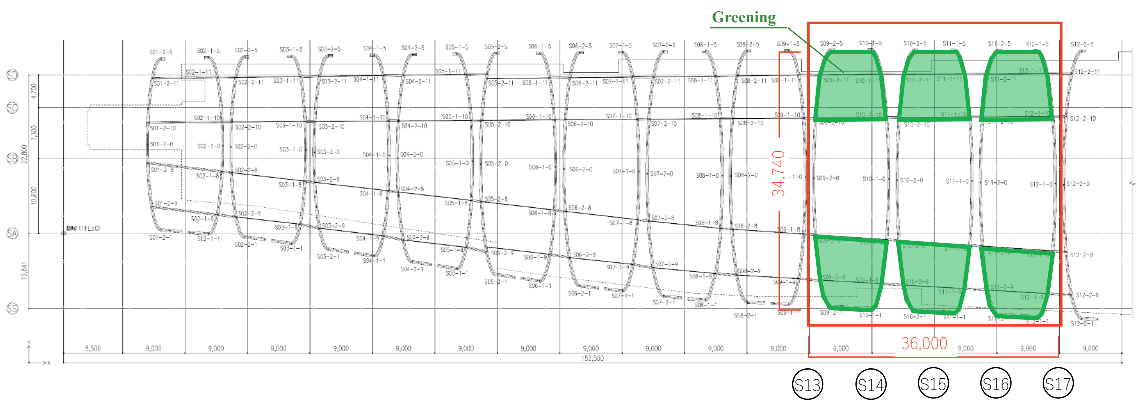


Fig. 11 – Area of calculate on the horizontal seismic coefficient

(1) The canopy should be treated as one layer, assuming that the layer's height is measured from the top surface of the support of the main structure at the roof floor level to the center of gravity of the canopy.

(2) The horizontal stiffness for eigenvalue analysis should be set, assuming that the deformation will be 1/100 when the canopy bears a horizontal force of 0.5 G.

(3) In the above range, the structural steel's weight is 700 kN, and the greenery weight is 125 kN (including wire mesh for greening 120 N/m^2).

(4) The weight of the rooftop structure for calculation of a horizontal seismic coefficient should be 850 kN, on the basis of the calculation of 700 + 125 = 825 kN.

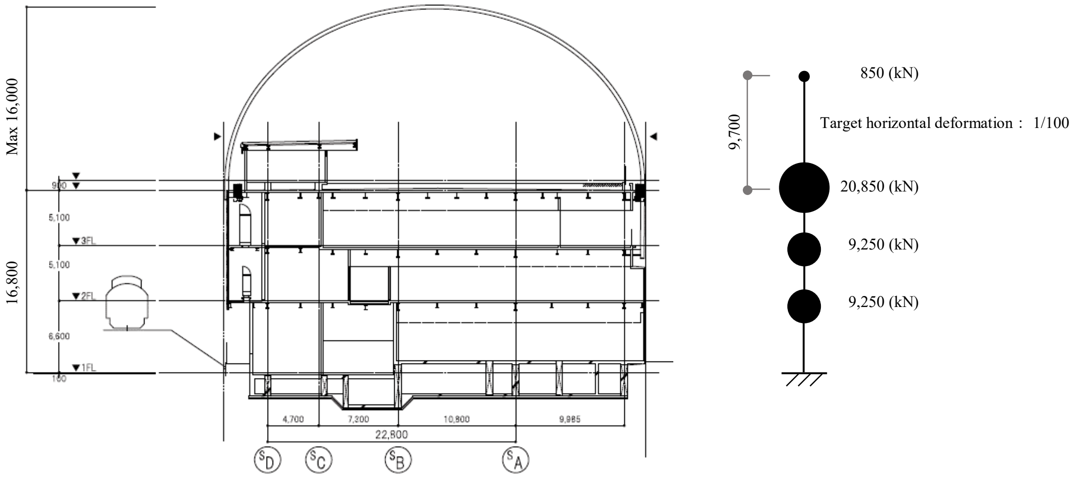


Fig. 12 – Structural analysis model

Assuming that the gross weight (W) of the canopy in the said range is 850 kN, and that the average height of the center of gravity (h) is 9.7 meters, the displacement (δ) caused by the horizontal deformation of 1/100 will be 9.7 cm. Consequently, the canopy's horizontal stiffness (K) is determined in the following equation:

$$K = 0.5W / \delta = 425 / 9.7 = 43.8 \text{ (kN/cm)} \quad (1)$$

Then the story shear coefficients by the floor levels in the range shown in Fig. 11 were calculated. Table 1 shows the properties of the floors and rooftop canopy of the main building. Eigenvalue analysis was performed by modelling the properties by the floors into a multi-mass equivalent shear model on the conditions listed in Table 1. The first natural period was 0.954 sec. in the X-direction and 0.949 sec. in the Y-direction. The story shear coefficient for each floor level was calculated by using the obtained first natural period. Table 2 shows the results.

Table 1 – Properties of the floors and rooftop canopy (1st)

Floor leve	Story height h (m)	Story stiffness for calculation C'i	Average weight W (kN/m^2)	Weight of each floor M (kN)	Story shear force Q (kN)	Average story drift angle R (rad)	Average relative story displacement δ (mm)	Story stiffness K (kN/cm)
Canopy	9.7	0.500	-	850	425	1/100	97.0	44
3	5.1	0.275	17.360	20,850	5,964	1/370	13.8	4,327
2	5.1	0.239	7.400	9,250	7,392	1/330	15.5	4,783
1	6.6	0.200	7.400	9,250	8,035	1/370	17.8	4,504

Table 2 – Result of analysis for Story shear coefficient (1st)

Floor leve	Base shear coefficient C0	αi	First natural period T1	Story shear distribution coefficient Ai	Vibration characteristic coefficient Rt	Seismic zoning coefficient Z	Story shear coefficient Ci
Canopy	-	0.02	0.935 (sec)	4.37	1.00	1.00	0.88
3	-	0.54		1.40			0.28
2	-	0.77		1.18			0.24
1	0.20	1.00		1.00			0.20

Similarly, the horizontal seismic coefficient was found by correcting the horizontal stiffness so that the deformation might be 1/100 when the horizontal seismic coefficient was 0.88 G as follows:

$$K = 0.88W / \delta = 748 / 9.7 = 77.1 \text{ (kN/cm)} \quad (2)$$

The story shear coefficient for each floor level was calculated by using the obtained horizontal stiffness. The first natural period was 0.870 sec. in the X-direction and 0.856 sec. in the Y-direction. The story shear coefficient for each floor level was calculated by using the obtained first natural period. Table 3 shows the results. The story shear coefficient for each floor level was calculated by using the obtained horizontal stiffness. Table 4 shows the results.

Table 3 – Properties of the floors and rooftop canopy (2nd)

Floor leve	Story height h (m)	Story stiffness for calculation C'i	Average weight W (kN/m^2)	Weight of each floor M (kN)	Story shear force Q (kN)	Average story drift angle R (rad)	Average relative story displacement δ (mm)	Story stiffness K (kN/cm)
Canopy	9.7	0.880	-	850	748	1/100	97.0	77
3	5.1	0.261	17.440	20,936	5,686	1/370	13.8	4,125
2	5.1	0.234	7.270	9,082	7,223	1/330	15.5	4,674
1	6.6	0.201	7.400	9,244	8,062	1/370	17.8	4,520

Table 4 – Result of analysis for Story shear coefficient (2nd)

Floor leve	Base shear coefficient C0	αi	First natural period T1	Story shear distribution coefficient Ai	Vibration characteristic coefficient Rt	Seismic zoning coefficient Z	Story shear coefficient Ci
Canopy	-	0.02	0.870 (sec)	4.30	1.00	1.00	0.86
3	-	0.54		1.40			0.28
2	-	0.77		1.18			0.24
1	0.20	1.00		1.00			0.20

The horizontal seismic coefficient was found by correcting the horizontal stiffness of the canopy assuming that the horizontal displacement at its center of gravity is 1/100. If the displacement caused by the obtained horizontal seismic coefficient is 1/100 or less, the first natural period will be shorter, which will cause the story shear coefficient generated at the canopy not to exceed 0.86 shown by Table 4.

Thus, it is assumed that the allowable deformation angle in the seismic design is about 1/100, with the horizontal seismic coefficient 0.86 G to be used in the primary design of the canopy. It was considered that we were able to properly evaluate the increased response according to the mass and stiffness ratios to the structure below because the above horizontal seismic coefficient is equivalent to 3.2 times the shear coefficient generated on the third floor of the structure below.

The comparison with the values as the result of the seismic response analysis is shown below for reference, which indicates that if the shear coefficient on the ground floor is standardized 0.2, the horizontal seismic coefficient as the result of the eigenvalue analysis is larger, from which it is clarified that the results in this study are based on a conservative decision.

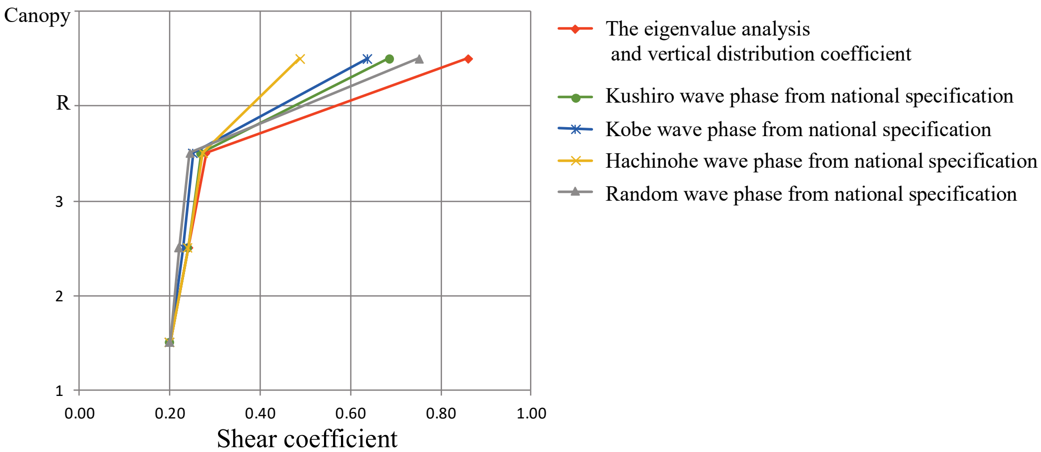


Fig. 13 – Shear coefficient distribution

5. Streamlining of Member Fabrication and Joint Details

In fabricating this arch configuration with steel tube, we needed to keep the curvature of each component constant. Therefore the curves carved out of a 3D shape as shown below were transformed into arcs to streamline the member fabrication process. For the transformation into arcs, the gaps at the angle conversion points were set to no larger than ±4 degrees which were allowable in design as well as in welding fabrication, and consequently we were able to group the arcs into 8 types in total.

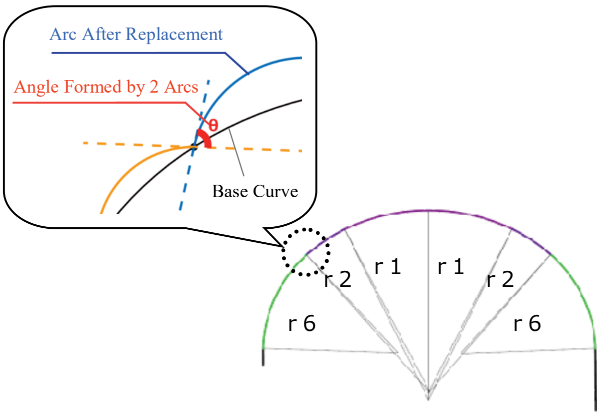


Fig. 14 – Transformed into Arcs

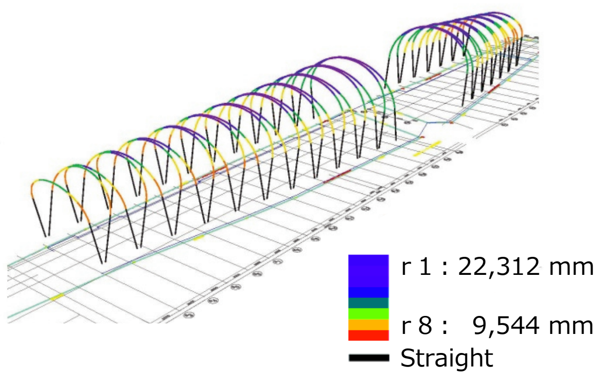
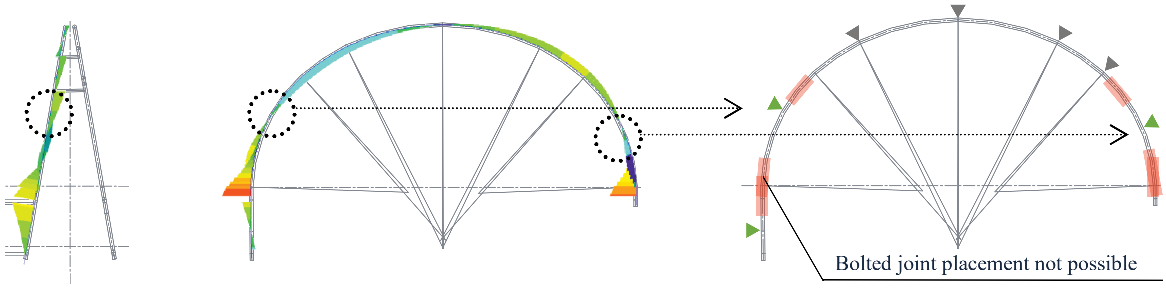


Fig. 15 – Grouping the arcs into 8 types

The arch configuration determined in this way was fabricated by using plated circular steel tubes. However, the detail of their joints at the site had to be improved to achieve this smooth configuration with plated steel tubes.

In the case of this project, site welding would have resulted in repair traces that spoil the appearance, and furthermore would have affected the rust-preventive performance of the inside of the hot-dip galvanized steel tubes due to the heat input. Hence, we considered use of bolted connections which do not require repair. Application of bolted connections raised two issues: deterioration of the design and strength. We focused on the measures against these issues and were successful in replacing all the site joints by bolted connections.

For the countermeasure taken against a decline in the strength was to locate joint positions where smaller stresses were generated, such as the positions of points of counterflexture caused by horizontal forces in an earthquake, which enabled replacement of all the site joints by bolted connections.



Stress Diagram – X Stress Diagram – Y Bolt joint location

Fig. 16 – Rule for laying Bolt joint out

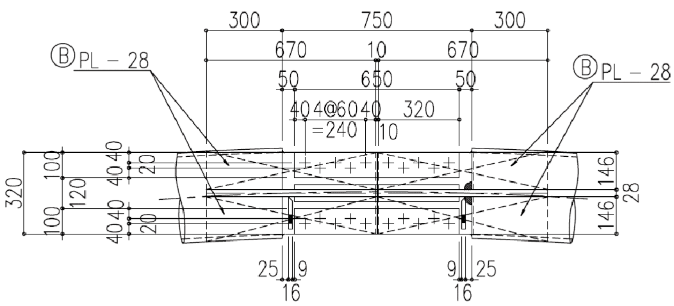


Fig. 17 – Production drawing

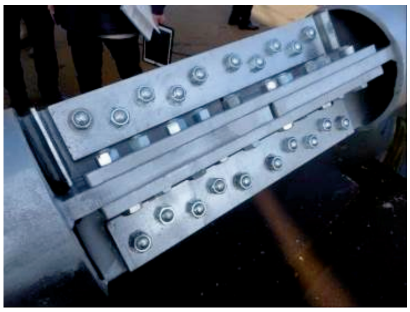


Fig. 18 – Bolted connections

For the design, the appearance of the canopy when looked up from the park below was considered, which has led to creation of an idea to prevent the cover screws from being visible when looked up by developing a cover plate in the shape of a 3/4 arc.

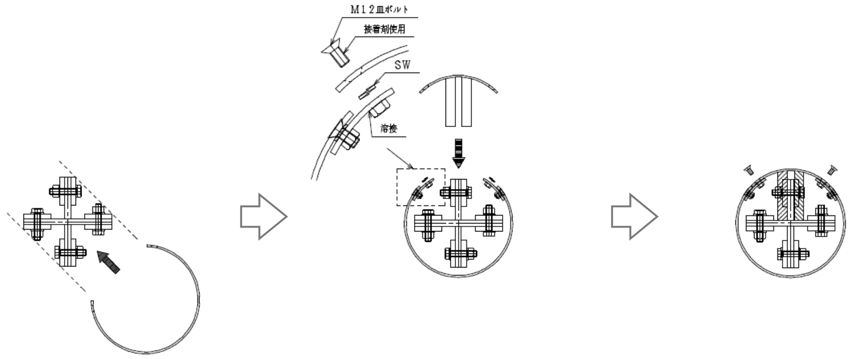


Fig. 19 – How to install cover plate

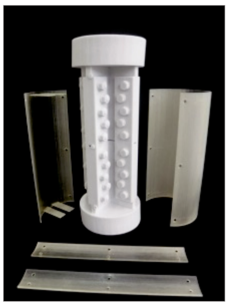


Fig. 20 – Mockups

Fig. 21 – Installation of joints

Fig. 22 – Construction of whole structure

6. Conclusion

The framing structure of the greening canopy covering the whole site consists of repeated elliptical shapes of different sizes ranging between 9.0 and 16.0 meters in height and between 25.0 and 35.6 meters in span. We determined the proper seismic force which would be generated on the rooftop structure and designed a continuous series of free-standing, A-shaped combinations of elliptical frame members, which enabled the structure to be fabricated of circular steel tubes, ranging between 9 mm and 32 mm in thickness, the diameters of which are all 356.6 mm.

Compact bolt joints are covered with circular steel tubes which are 356.6 mm in diameter like the main tubes. Thus a seamless structural frame is realized.

7. Acknowledgements

The land owner Shibuya Ward government and the project owner Mitsui Fudosan Co., Ltd. have extended tremendous support to us towards the realization of this project. The authors hereby express their sincere gratitude to those parties concerned.

