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Feature Article: Fire Resistant Design of Steel Structures

Membrane Actions of Reinforced Concrete Floor Slabs Exposed to Fire

Serial Article: Latest Design of Steel Buildings in Japan (5)

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Membrane Actions of Reinforced Concrete Floor Slabs Exposed to Fire

by Takeo Hirashima, Professor, Chiba University

Takeo Hirashima: After completing the course of graduate school, Chiba University, he entered Fujita Corporation as a researcher in 1996. He became Research Associate, Faculty of Engineering of Chiba University in 2001 and Associate Professor in 2006. He assumed his current position as Professor, Department of Architecture, Graduate School of Engineering, Chiba University in 2015.

Part 1: Application to Fire Resistant Design of Steel Structures

Fire Resistance Required in Building Construction

Buildings are required to possess fire resistance that conforms to the application purpose and scale of the building. Specifically, in an event of fire, the wall and floor slab applied as compartment elements are required to possess thermal insulation and fire integrity, and further the column, beam, floor slab and bearing wall are required to possess appropriate load-bearing capacity. Accordingly, it is general practice in steel-structure construction to provide fire protection with columns and beams and to ensure fire resistance. In Japan, fire protection is provided with the secondary beam that supports the floor slab.

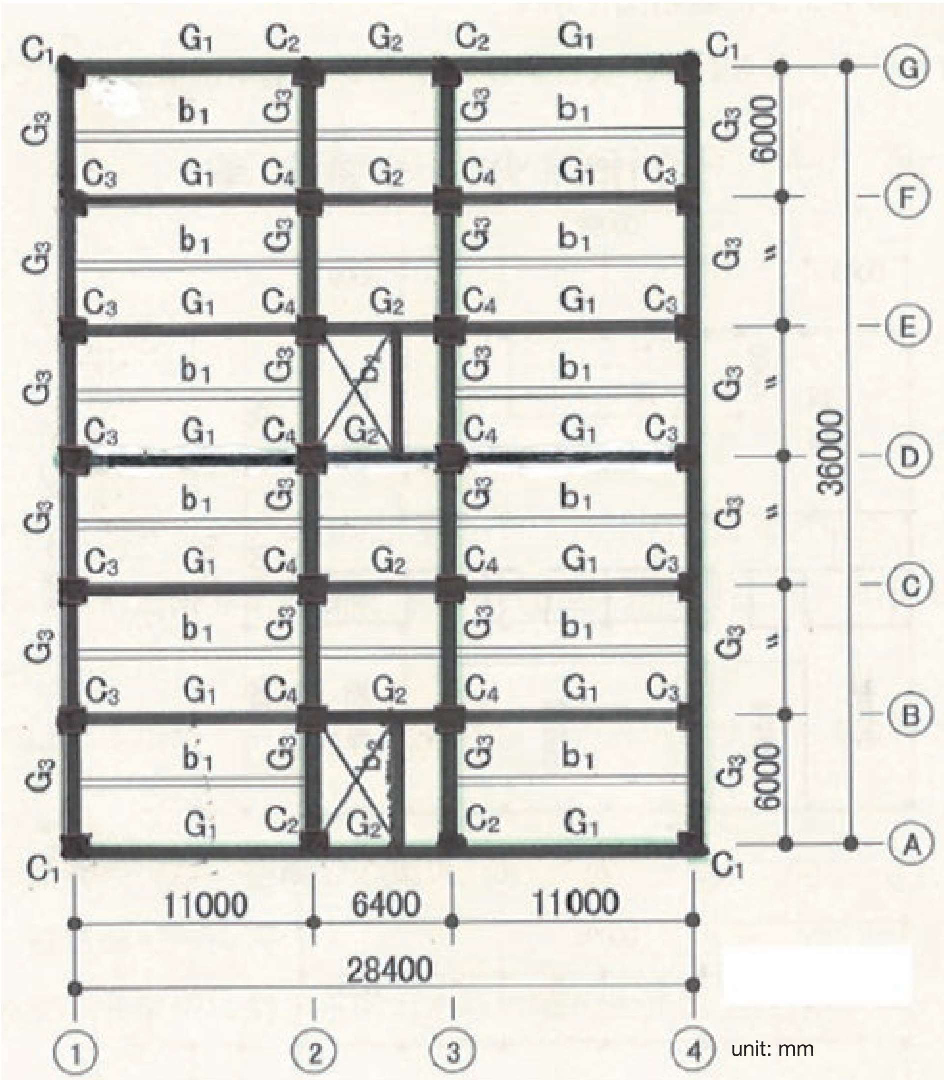
In fire resistance tests for deck composite slabs designed as 1-way slabs, because of the precondition that the steel beam to support the slab is to be applied in a sound condition, a secondary beam is provided with fire protection. Meanwhile, in the floor slabs that support the load as 2-way slabs, as the deflection increases, the membrane stress increases, and as a result, the ultimate strength greatly surpasses the bending strength due to this membrane action. It is reported in past experimental studies[1)-4)] that even when the floor slab was exposed to fire for 75~180 minutes, it continued to support the load.

Accordingly, in cases when the load bearing capacity of composite floor slabs during fire is assessed taking into account the membrane action, it is feasible to eliminate the application of fire protection for secondary beams. If an unprotected secondary beam (beam with no fire protection covering) can be applied in steel-structure construction, great effects will be brought about such as reduced construction cost, enhanced freedom in design and reduced waste disposal at the stage of dismantling buildings.

In steel-structure construction in earthquake-prone Japan, moment frame structures are being increasingly applied in which the column is rigidly joined with the primary beam. Accordingly, in making designs that take into account the membrane action of the floor slab, it is required that the peripheral beams to support the floor slab, including the joint, are applied in a sound condition. In steel structures employing moment frames in Japan, because the rigid joint is applied for primary beam-end connection, it is considered that designs can easily be made that take into account the membrane action of the floor slab.

Fig. 1 shows an assumed framing plan

Fig. 1 An Example of Framing Plans in the Case in Which Secondary Beams (b_1) Are Not Covered with Fire Protection



for a standard floor in an example of the design of a steel-structure office building[5]. If the 11×6 m floor slab can support the load during fire by utilizing the membrane action, the secondary beam (fine dual lines; symbol: b_1) shown in the framing plan is likely to be made unprotected. In the figure, the primary beam located in the painted thick line serves as the vertical-direction supporting member for the floor slab, and the floor slab surrounded by these supporting members and the unprotected secondary beam work as one composite flooring system.

Targeting such composite flooring systems, load-bearing fire test and numerical analysis were conducted to examine the membrane action of reinforced concrete floor slabs during fire. In the succeeding Part 2 and Part 3 of the current issue, part of the results of the examinations of membrane actions obtained in the above-mentioned test and analysis are introduced.

In Part 4, the fire resistant design of steel structures in Japan and its future development are introduced as a final topic of this serial article.

Purpose of the Study of Fire Resistance of RC Flooring Systems

In spaces where the fire range is restricted and the steel product temperature becomes low (for example, open-type mobile parking lots, atrium spaces, space inside elevator shafts), there are cases in which unprotected steel members can be applied by conducting the fire resistant design of the steel structure. Further, even in spaces where fully-developed fires may occur, if the use of fire protection for the secondary beams of steel structures can be eliminated, more rational design becomes possible, and as a result the predominance of steel structures over other structural systems will grow.

To attain this goal, it is necessary to conduct fire resistant design for steel structures that takes into account the membrane action of floor slabs during fire, and at the same time, it is also necessary to accumulate basic data relating to such fire resistant design.

With regard to past experimental studies of the membrane action of RC floor slabs in an event of fire, the compartment fire test conducted at a full-scale steel-frame building of the Cardington Laboratory in the UK in the 1990s is well known[1]. In the test, a composite flooring system employing unprotected secondary beams and composite slabs was exposed to a fully-developed fire in an office space, but the system continued to support the load while being accompanied with large deformation. Further, it was explained by means of numerical analysis that this phenomenon is caused by the tensile membrane action of the floor slab[6], based on which fire resistant design method for the composite flooring system was proposed that takes into account this tensile membrane action[7]. Since then, many researchers have tackled this appealing research theme.

Meanwhile, it cannot be said that thorough experimental studies on the fire resistance of full-scale flooring systems have ever been made, as shown in Table 1. This was due primarily to both the constraints on relevant experimental facilities and the cost.

In addition, any experimental study on the membrane action of floor slabs during fire had not been conducted until recently in Japan, and currently it is impossible to perform fire resistant design that takes into account the membrane action during fire.

Given such a situation, in order to clarify behavior of composite flooring systems composed of flat RC slabs and deck composite slabs in an event of a fire, load-bearing fire test and numerical analysis were conducted for the composite flooring system. Because the test and analysis results particularly pertaining to the composite flooring system employing flat concrete floor slab and unprotected steel beam (beam with no fire protection covering) are less available in past experimental studies, the test and analysis were made for this type of composite flooring system in this experimental study, the results of which are introduced

Table 1 Outline of Previous Experimental Studies and This Study on Composite Flooring Systems

Ref.	Floor slab	Specimen size (mm)	Unprotected beam	Total load (kN/m²)	Heating duration (ISO 834)	Result of the maximum vertical displacement	Reinforcement temperature
1)	Composite slab	6000×9000 Depth 200 (130+70)	356x171	–	Under real fire condition	428 mm (Test 3)	–
						640 mm (Test 6)	
2)	Flat RC slab	3300×4300 Depth 100	None	5.31	180 min	210 mm	630 – 750°C
				5.51		155 mm	
				5.32		271 mm	
	Composite slab	3300×4300 Depth 130 (75+55)		5.43		253 mm	500 – 700°C
3)	Composite slab	6660×8735 Depth 155 (97+58)	300x150 (Two beams)	3.87	120 min	About 450 mm	280 – 350°C
4)	Composite slab	3720×5232 Depth 146 (70+76)	250x118	18.38	75 min	175 mm	Ave. 17°C
				17.71	90 min	140 mm	Ave. 240°C
			None	8.75	100 min	135 mm	Ave. 370°C
				9.47	100 min	150 mm	Ave. 360°C
This study	Flat RC slab	4200×5800 Depth 120	350x175	11.23	132 min	190 mm	500 – 550°C
					202 min	269 mm	670 – 710°C
			None		217 min	280 mm	740 – 780°C
	Composite slab	4200×5800 Depth 130 (80+50)	350x175	8.04	210 min	350 mm	500 – 610°C
				10.72	156 min	370 mm	420 – 550°C

in Part 2 and Part 3. The load-bearing fire test and numerical analysis were subsidized by the Japan Iron and Steel Federation.

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Part 2: Load-bearing Fire Tests for Composite Flooring Systems

Aim of Load-bearing Fire Tests

In Part 2, the results of load-bearing fire test are introduced that targets a composite flooring system composed of flat RC slabs and unprotected beams.

When the unprotected steel beam applied as a secondary beam is exposed to fire, it loses strength at an early stage of fire. On the other hand, for flat RC slabs supported in 2 ways, when deflection increases, membrane stress occurs within the cross section of the slab, and as a result the load bearing capacity of the slab is improved due to this membrane action. In RC slabs subjected to heating from the lower side during fire, because of large deflection caused by reduction in the rigidity of the material and thermal deflection due to the temperature difference between the upper and lower sides of slab, the membrane action becomes more striking at that stage of fire than at the room temperature stage, and as a result the slab can withstand fire heating for a long time.

The ultimate load of an RC slab during fire surpasses by a great margin the bending collapse load based on yield-line theory, and the ratio of the ultimate load to the bending collapse load depends on the amount of deflection[1]. Further, a characteristic feature of the membrane action of RC slab lies in that compression stress occurs to the circumferential direction of the peripheral section of the RC slab and this compression stress balances the tensile stress widely occurring at the center section of slab, and as a result it is unnecessary to restrict the horizontal-direction displacement at the four peripheral sections of the slab[2].

In RC slabs in which two-tier reinforcements are arranged, when subjected to fire heating, the temperature at the upper-side reinforcement becomes lower than that at the lower-side reinforcement, and thus the tensile strength at the upper-side reinforcement is kept to a high level for a long time. As a result, it is forecast that the retention of strength by the slab due to the membrane action during fire will become more conspicuous.

In order to understand these behaviors, a load-bearing fire test was carried out by means of standard fire heating in accordance with ISO 834, targeting a composite flooring system composed of two-tier reinforcement-arranged flat RC slabs and unprotected steel beams.

Outline of Testing

• Test Conditions

Table 1 shows the conditions of the load-bearing fire test. The first aim was to understand the effect of the headed stud arranged at the periphery of the slab on the deflection behavior of the RC slab during fire by comparing specimens No. 1 and No. 2. The second aim was to understand the effect of applying unprotected secondary beams on the fire resistance of the composite flooring system by comparing specimens No. 2 and No. 3.

In the test, a load of 11.23 kN/m^2, including the dead load of the floor slab, was given as the target value. The live load was the collapse load based on yield-line theory in the case when the RC slab was simply supported at the four peripheral sections of the slab. The collapse load was calculated using the following equation (1) paying no attention to upper-side reinforcement.

$$W_p = \frac{24 \times 0.9\, d\, A_s\, \sigma_u}{l^2} \left(\frac{n}{\sqrt{1+3n^2} - 1} \right)^2 \quad (1)$$

where,

W_p: Collapse load based on yield-line theory (=11.23 kN/m^2)
l: Short-side span (=4.2 m)
n: Aspect ratio (=1.38)
d: Effective depth (=0.09 m)
As: Cross section of lower-side reinforcement per unit width (=366.6 m^2/m)
σ_u: Yield stress of lower-side reinforcement shown in mill sheet (=0.380 kN/m^2)

The live load was set at 8.35 kN/m^2, assuming that a unit weight of concrete applied to the floor slab is 24 kN/m^3. The test specimen was heated based on the standard time/temperature curve in accordance with ISO 834-1.

Table 1 Parameters in Load-bearing Fire Tests

Specimen	Headed stud at the end of the slab	Unprotected secondary beam	Load including dead load of the slab	Heating condition
No. 1	Diameter:15.4 mm Distance: 150 mm	H-300×150×6.5×9 (SN400)	11.23 kN/m^2	Standard fire curve in accordance with ISO 834-1
No. 2	None			
No. 3		None		

Fig. 1 Outline of Specimen (Example: Specimen No. 1)

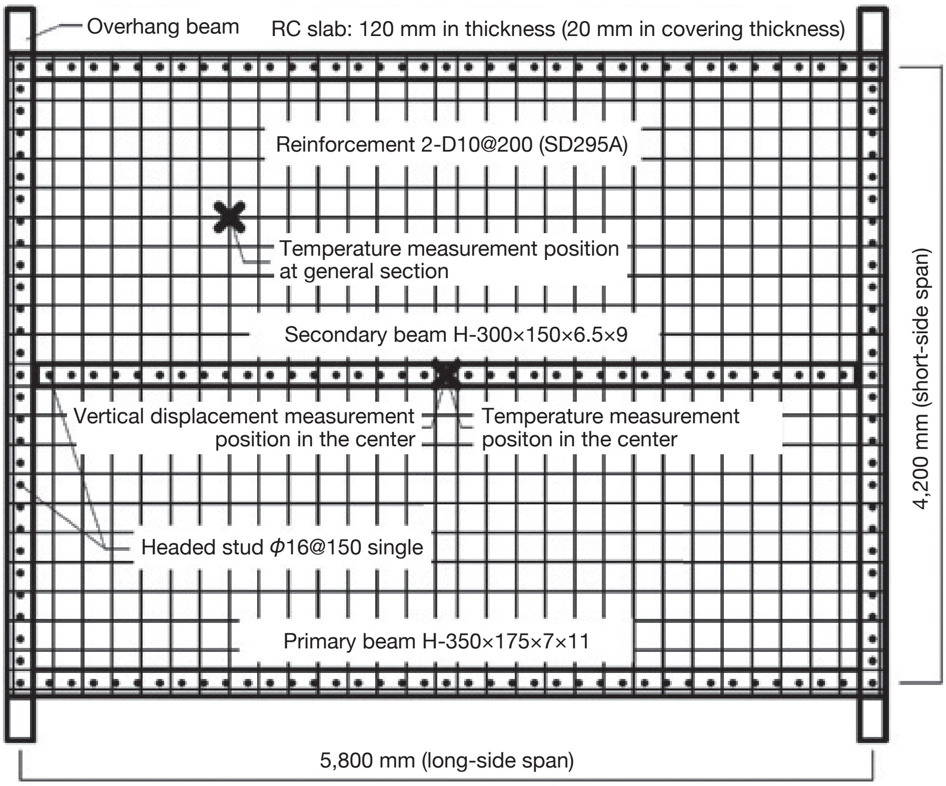



Photo 1 Specimen No. 1 before placement of concrete


Photo 2 Specimen No. 1 to which weights are arranged

• Test Specimens

Fig. 1 shows an outline of specimen No. 1. Photo 1 shows the specimen before the placement of concrete. The span of the flooring system specimen was set at 5.8×4.2 m in a distance between primary beam cores. The thickness of the flat RC slab was set at 120 mm, and a 1 mm-thick flat deck was applied as a concrete permanent form.

Regarding specimen No. 1, the headed stud was arranged in a primary beam that supported the four peripheral sections of the RC slab so as to restrict rotational- and horizontal-direction displacement at the slab edge under conditions roughly similar to those in a practical steel structure. The unprotected secondary beam was structured to a complete composite beam together with the RC slab. The cross section of the secondary beam was H-300×150×6.5×9, and the steel grade adopted was SN400B (JIS G 3136). The lower-limit yield point of the SN400B was 235 N/mm^2, and the lower-limit tensile strength was 400 N/mm^2. Three high-strength bolts (diameter: 20 mm) were arranged at the end of the secondary beam web to form a pin joint. The cross section of the primary beam applied for the peripheral section of the slab was H-350×175×7×11. The orthogonal primary beams were joined by means of full-penetration flange welding.

Regarding specimen No. 2, the same specifications as those of specimen No. 1 were adopted except for the headed stud arranged in the primary beam that supported the four peripheral sections of the floor slab.

Regarding specimen No. 3, the same specifications as those of specimen No. 2 were adopted except for the secondary beam arranged in the center section of the floor slab. That is, specimen No. 3 had a structure in which the RC slab was only arranged on the primary beam.

Deformed steel bars (nominal diameter: 10 mm) were applied for RC slab reinforcement and arranged at 200 mm intervals between reinforcing bars for both the upper- and lower-side reinforcements and with a covering thickness of 20 mm. According to the mill sheet for the deformed steel bar applied, its yield stress was 380 N/mm^2. The water to cement ratio of the concrete was set at 51.9%, and polypropylene fiber was mixed with the concrete to prevent explosions in the case of fire. The strength of the concrete after 28-days aging was 36.1 N/mm^2. A ceramic-wool blanket with a thickness of 100 mm for use as fire protection was provided to the heating side of the peripheral primary beams, and the same fire-protection blanket with a thickness of 25 mm was provided to the high-strength bolt joints at the end of the secondary beam.

• Test Methods

The test was conducted using a heating furnace for horizontal elements operated by the Building Research Institute. The heating area at the lower surface of the RC slab was set at 5.6×4.0 m. A total of 24 weights (6×4 weights) were placed at equal intervals on the upper surface of the RC slab, and the slab was then heated from its lower surface. The weight per weight was set at 8.62 kN. Photo 2 shows the specimen on which the weight was placed.

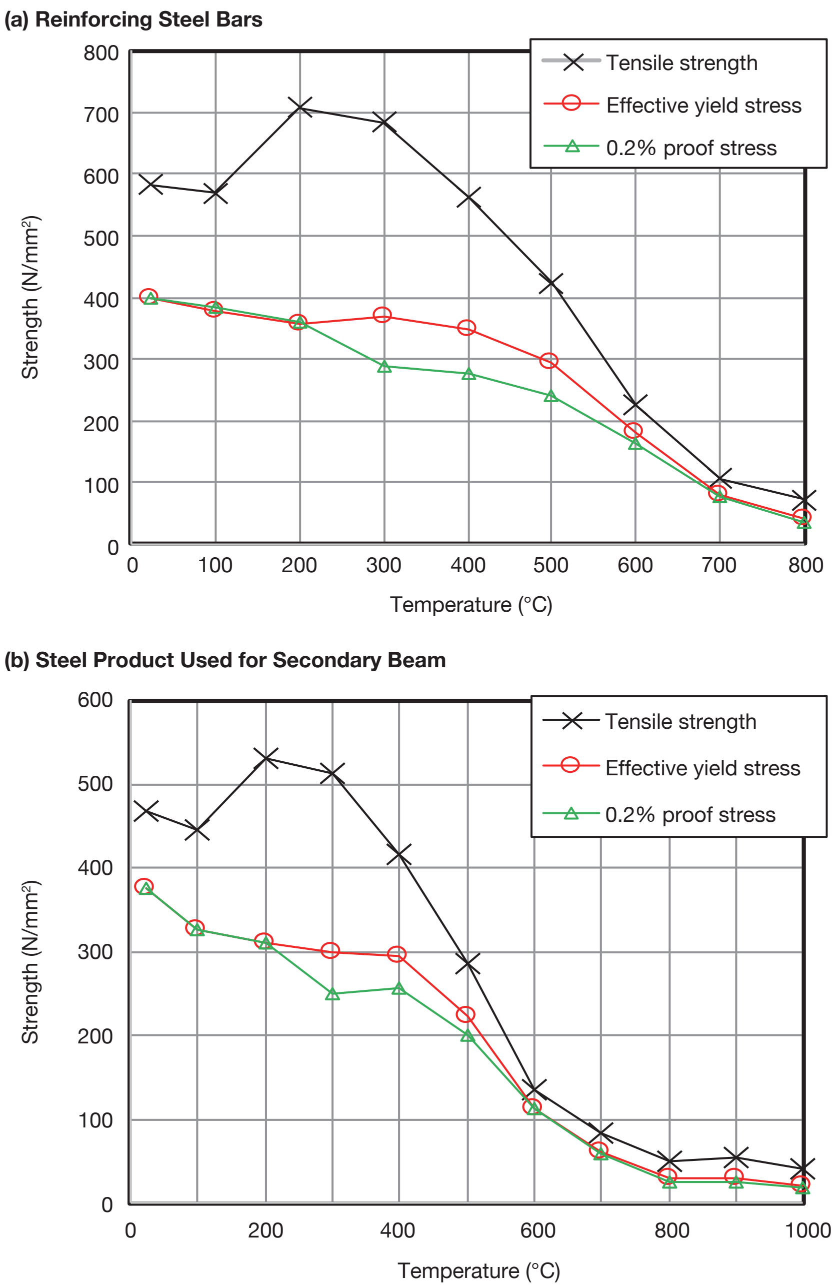
The vertical displacement of the slab was measured at 18 positions at the center section of the short and long sides and on the diagonal line. The temperature of the unprotected secondary beam was measured at the center section and at the end of the bolt. The temperature within the cross section of the RC slab was measured at the center section and at a position at a distance 1/4 of the diagonal line, as shown in Fig. 1. The temperature of reinforcement within the RC slab was measured at positions where the long- and short-side reinforcement bars crossed or contacted each other.

• High-temperature Mechanical Properties of Steel Products Applied

The high-temperature tension test based on JIS G 0567 was conducted for reinforcing bars and secondary beams used for the specimens in the current load-bearing fire test. Fig. 2 (a)-(b) show the relationship between the 0.2% proof stress, stress at 1% strain and tensile strength and the temperature, which was obtained from the test results. The effective yield stress shown in the figure is the value for the stress at 1% strain.

As shown in Fig. 2 (a), the strength of the reinforcing steel bars was similar to that at room temperature before the temperature reached 400°C, but the strength rapidly lowered when the temperature surpassed 400°C, and when the temperature reached 700°C, the strength lowered to a level of 1/4 to 1/5 that at room temperature. The trend of strength lowering was nearly close to the high-temperature strength retention rate of hot-rolled reinforcing bars in Eurocode 2[3].

Fig. 2 High-temperature Strength of Steel Products Used for Specimens



As shown in Fig. 2 (b), the strength of the steel member used as the unprotected secondary beam lowered to a level of nearly 1/10 that at room temperature when the temperature surpassed 800°C. It is forecast from these experimental test results that unprotected secondary beam subjected to standard fire heating would lose its strength in an earlier stage of a fire.

Test Results

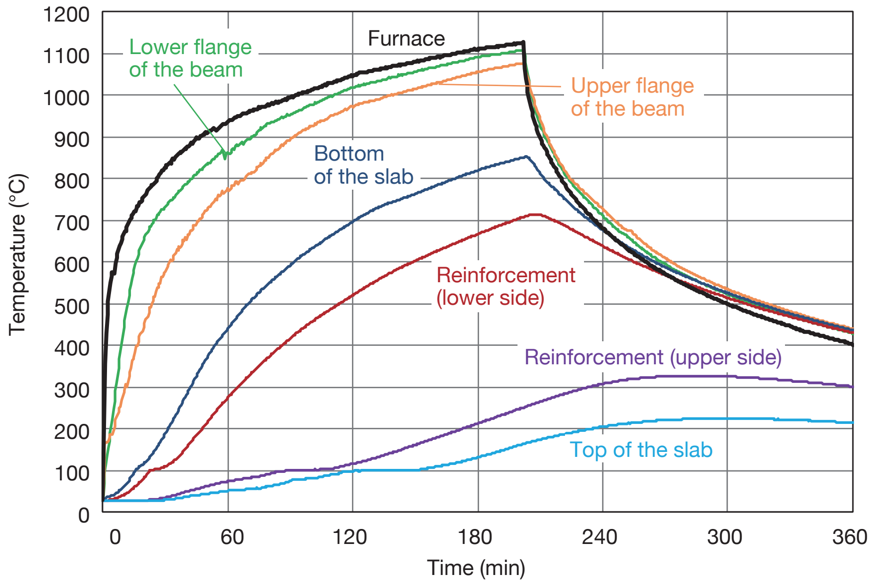
• Temperature Measurement Results

Fig. 3 shows the temperature measurement results for specimen No. 2 as an example of the temperature measurements results for the composite RC flooring system. In the test of specimen No. 2, heating stopped at the stage when the temperature of the lower-side reinforcement in the slab reached 700°C. The heating duration was 202 minutes. The temperature inside the heating furnace was controlled following the standard fire curve in accordance with ISO 834-1.

The temperature of the lower flange of the unprotected beam surpassed 700°C after 30 minutes of heating, and 1,000°C after 120 minutes of heating. The temperature of the web reached a level similar to that of the upper flange. The temperature of the upper flange came closer to that of the lower flange as the heating duration increased, and it surpassed 950°C after 120 minutes of heating. The temperature of the RC slab lower surface was low in comparison with the temperature inside the furnace, and it reached about 800°C after 120 minutes of heating. For the test specimens in this test, a flat deck was applied as a permanent form. Therefore, the lower surface of the RC slab was not exposed directly to flames, and a gap occurred between the flat deck and the RC slab lower surface during heating, and as a result it was considered that the temperature of the RC slab lower surface was lower than that inside the furnace.

While the temperature of the lower-side reinforcement in the RC slab was low, or about 250°C after 60 minutes of heating, due to stagnation from the temperature rising in the vicinity of 100°C accompanied by the evaporation of water contained in the concrete, the temperature then increased as the heating duration lapsed, reaching about 650°C after 180 minutes of heating. On the other hand, the temperature of the upper-side reinforcement in the RC slab showed a longer stagnation time from the rising temperature at 100°C compared to the lower-side reinforcement, and it was 200°C even after 180 minutes of heating. The temperature of the RC slab upper surface was about 120°C even after 180 minutes of heating, and the difference in temperatures between the upper and lower-surfaces after 180 minutes of heating was about 700°C.

Fig. 3 Temperature Measurement Results (Specimen No. 2)



• Deflection at the Center Section of RC Slabs

Fig. 4 shows changes in deflection occurring in the center section of the RC slab.

The heating duration was 132 minutes for specimen No. 1, 202 minutes for specimen No. 2 and 217 minutes for specimen No. 3. Because specimen No. 1 was first subjected to the test, heating was stopped when the temperature of the lower-side reinforcement reached 500°C. When the damage condition of specimen No. 1 was confirmed, it was found that the RC slab concrete caused no explosion and that the damage to the high-strength bolt joint at the secondary beam end was lessened. Consequently, heating was sustained for specimen No. 2 until the temperature of lower-side reinforcement reached 700°C, and in the test for specimen No. 3 heating was sustained until the heating duration reached 210 minutes.

In the case of specimens No. 1 and No. 2, large deflection occurred for 30 minutes from the start of heating. This large deflection was greatly affected by the thermal deflection that occurred due to the difference of temperatures between the upper and lower flanges of the unprotected secondary beams. In specimen No. 3, deflection rapidly increased for about 2 minutes after the start of heating to reach about 50 mm. The reason for this rapid increase was considered to be attributable to the cracking that occurred in the lower-side concrete due to the thermal expansion of the flat deck plate and the movement of the neutral axis in the cross section of the RC slab to the upper-surface side of the slab. Meanwhile, in specimen No. 3, which did not use a secondary beam, there was deflection for 30 minutes after the start of heating, which was smaller than that of specimens No. 1 and No. 2.

After 40 minutes from the start of heating, deflection increased in every specimen nearly in proportion to the increase in heating duration, and they showed nearly identical deflections. In spite of whether or not the headed stud was arranged at the peripheral area of the slab, both specimens No. 1 and No. 2 showed nearly identical deflection behaviors, and therefore it was found that the effect of the restraint of rotational and horizontal displacement at the slab end on the deflection behavior of composite flooring systems during fire was

Fig. 4 Deflection in the Center of Beam

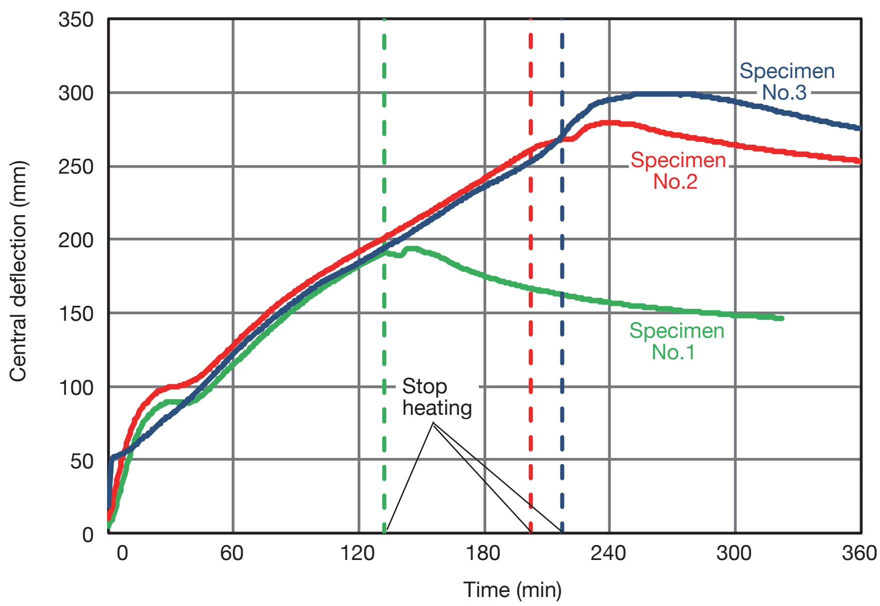


Photo 3 Cracking conditions on upper surfaces of RC slabs in specimens No. 1, No. 2 and No. 3

(a) Specimen No. 1

(b) Specimen No. 2

(c) Specimen No. 3

small. Further, in spite of whether or not the unprotected secondary beam was applied, both specimens No. 2 and No. 3 showed nearly identical deflection behaviors after 40 minutes from the start of heating, and therefore it was considered that the live load of the composite flooring system was supported only by the RC slab after 40 minutes from the start of heating. Further, these test results can clearly be understood from that the lower flange temperature surpassed 800°C after 40 minutes from the start of heating in the temperature measurement results in Fig. 3 and that the strength of the secondary beam at 800°C lowered to nearly 1/10 that at room temperature as shown in Fig. 2 (b).

The maximum deflection of specimens No.2 and No. 3 after stopping heating was 1/15 the deflection of the short-side span of the RC slab. Because large deflection occurred in the slab, the membrane stress occurred within the slab, and it was considered that the slab could sustain the load due to this membrane action. In addition, because the temperature of the upper-side reinforcement was low, it was considered that the tension resistance of the upper-side reinforcement worked effectively in terms of membrane action. Further, these test results can be understood by comparing the temperature of the upper and lower-side reinforcements after 3 hours from the start of heating shown in Fig. 3 and the high-temperature strength of reinforcing steel bars shown in Fig. 2 (a).

While heating was stopped for specimen No. 1 after 132 minutes, when taking into account the test results for specimens No. 2 and No. 3, it was considered that specimen No. 1 sustained the load-bearing capacity for 3 hours or more as with the case of specimens No. 2 and No. 3.

• Crack Occurrence Condition at Upper Surface of RC Slab

Photo 3 shows the occurrence of cracking on the upper surface of the RC slab.

In specimen No. 1 shown in Photo 3 (a), because the heating duration was short, the cracking width in the center section of the RC slab was narrow, but because the end of the RC slab was restricted, it showed remarkable cracking in the vicinity of the end.

In specimen No. 2 shown in Photo 3 (b), large cracks vertical to the long-side direction occurred in the center section of the RC slab. In the center section of the RC slab, it was considered that the full cross section experienced a tension region in the long-side direction area. Meanwhile, no cracking vertical to the short-side direction was observed. Because the curvature in the short-side direction area was large, it was considered that the compression due to bending and the tension due to membrane stress were offset at the upper surface of the slab.

In specimen No. 3 shown in Photo 3 (c), the cracking condition was similar to that of specimen No. 2.

In specimen No. 1, oval-shaped cracking was observed in the vicinity of the peripheral section. In specimens No. 2 and No. 3, diagonal cracking like diamond-shaped cracking was observed. The reason for such cracking was considered to be the suppression of floating of the RC slab at the corner section by the weight arranged in the corner section. At the stage where the membrane action of the RC slab worked, the tension force occurred to the radial direction in the center section of the RC slab, and the compression force occurred to the circumferential direction in the peripheral section of the RC slab.

The trends in cracking in the RC slabs under the condition of the membrane action were confirmed from the cracking conditions in these three specimens used in this load-bearing fire test for the composite flooring system.

Potential for Unprotected Application of Secondary Beams

The following will summarize the results of the load-bearing fire test that was conducted for the composite flooring system employing the two-tier reinforcement-arranged flat RC slabs and unprotected steel beams:

- The RC slab to which the bending collapse load based on the yield-line theory was given sustained load-bearing capacity even after being subjected to 3.5 hours of standard fire heating while involving deflection surpassing 1/20 that of the short-side span.
- In the RC slab in which two-tier reinforcements were arranged, even when the temperature of the lower-side reinforcement reached 700°C, the temperature of the upper-side reinforcement was about 300°C, thereby allowing for the slab to retain its load-bearing capacity. The reason for this successful retention was considered to be that the tension resistance of the upper-side reinforcement worked effectively due to the membrane action of the RC slab.
- In the composite flooring system used in this test, the effect of the application of unprotected steel beam on fire resistance was small.
- The effect of the headed studs arranged in the four peripheral sections of the RC slab on the deflection behavior of the RC slab during fire was small. Due to the small effect, even when the rotational and horizontal displacement at the end of the slab was not restricted, the membrane action of the RC slab worked effectively, and it was thus shown that the load-bearing capacity of the composite flooring system was retained during fire.
- It was shown from these test results that the application of unprotected secondary beam in the composite flooring system is made possible by means of the membrane action of the RC slab.

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Membrane Actions of Reinforced Concrete Floor Slabs Exposed to Fire

by Kei Kimura, Steel Research Laboratories, Nippon Steel Corporation

Kei Kimura: After completing the course at the Graduate School of Engineering, Kyoto University, he entered Nippon Steel Corporation in 2009. Then he was assigned to the Steel Research Laboratories, Research & Development, and in 2014 became Researcher of Steel Structures Research Laboratory. He serves as a member of Sub-committee on Fire Resistant Design for Steel Structures, AIJ, and assumes his current position as Senior Researcher since 2019.

Part 3: Numerical Analysis of the Load-bearing Fire Tests for Composite Flooring Systems

Purpose of Numerical Analysis

A numerical analysis was made targeting the load-bearing fire test of composite flooring systems composed of flat RC slabs and unprotected beams (beams with no fire protection covering), and the test results and numerical analysis results were compared. The main aim in numerical analysis was to verify the accuracy of the numerical analysis approach, and at the same time to study the stress distribution within the composite floor surface at the stage where the membrane action worked, the data for which is difficult to obtain from the load-bearing fire test.

Analysis Targets

The numerical analysis targeted the load-bearing fire test for the composite flooring system introduced in the preceding Part 2. The plane size of the floor slab in the composite flooring system measured 5,800×4,200 mm with a thickness of 120 mm, and 10 mm-diameter reinforcing steel bars were arranged in the slab in two tiers at a pitch of 200 mm. Four floor perimeters were supported with protected primary beams, and unprotected secondary beams were arranged as they connected the center of the short sides of slab.

A uniformly distributed load amounting to 11.23 kN/m^2 (including the dead load) was loaded on the floor slab. The slab was heated from its lower surface by means of the standard fire curve in accordance with ISO 834.

Then the numerical analysis was made, targeting test specimen No. 1 in which headed studs were arranged at the perimeter beams.

Outline of Numerical Analysis Approach

For the numerical analysis, the finite element analysis software "SAFIR" developed at the University of Liege in Belgium was adopted. This software is composed of a thermal analysis program and a structural analysis program.

First, the distribution and hysteresis of the temperature in the cross section of the structural members were calculated using the thermal analysis program, and then the stress and deformation conditions of these members were calculated using the structural analysis program that employs the results thus obtained in the above calculation. (For more details about SAFIR, refer to reference 1).

Thermal Analysis

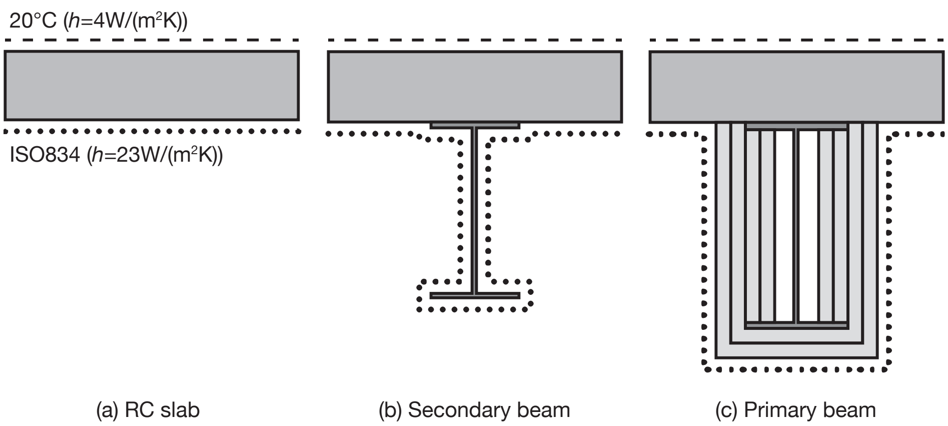
• Analysis Models

Fig. 1 shows the thermal analysis model. Fig. 1 (a) shows the RC slab, (b) the secondary beam and (c) the primary beam. For the thermal analysis model in Fig. 1, the thickness of the RC slab was set at 120 mm, the cross section of the secondary beam H-300×150×6.5×9, and the cross section of the primary beam H-350×175×7×11. In Fig. 1 (c), ceramic wool was coated on 3 surfaces of the H beam, excluding its upper flange, and the heat transfer caused by convection and radiation in the cavity was taken into account.

The thermal properties of the concrete and steel followed that prescribed in Eurocode 4[2)], and the moisture content of the concrete was set at 5.8% based on the material test results. Further, the thermal properties of the ceramic wool applied the room-temperature value of calcium silicate board, and the moisture content was set at 10%.

In order to simulate the condition in which heating at the flashover stage of fire is applied to respective structur-

Fig. 1 Thermal Analysis Model



al members from their lower side, the atmospheric temperature at the heated surface followed the standard heating temperature prescribed in ISO 834, and thus the atmospheric temperature at the non-heated surface was set at 20°C. The convection heat transfer coefficient at the heated surface was set at 23 W/(m^2・K), and that at the non-heated surface at 4 W/(m^2・K). Incidentally, reinforcing steel bars and headed studs in the RC slab, and steel decks were not modelled, and the effect of these materials on the member temperature was disregarded.

● Results of Analysis

Fig. 2 shows the temperature hysteresis of the RC slab as an example of the thermal analysis results. In this figure, the solid line shows the analysis values, and the test value of specimen No. 2, which was heated for 202 minutes, is plotted. Regarding the temperature of the lower-side reinforcement, a major factor affecting membrane action during fire, it was understood from the figure that the difference in temperature between the numerical analysis and the test was approximately 100°C at maximum.

On the other hand, while the temperature at the lower surface of the composite floor obtained by numerical analysis was higher by as much as 300°C than that obtained in the test, the reason for the higher temperature was attributable to that the thermal insulation effect of the steel deck removed from concrete and the air layer on the temperature was disregarded. Further, while the temperature at the upper-side reinforcement and the upper surface of the composite floor, obtained by numerical analysis, was about 50°C higher than that obtained by the test, the reason for this was attributable to the fact that the movement of moisture inside the composite floor could not be taken into account.

While the temperatures of the structural members obtained by numerical analysis showed a trend of being higher than those obtained by the test as seen above, it was considered possible to qualitatively assess these temperatures, and these numerical analysis results were thus adopted as the input data for use in structural analysis.

Structural Analysis

● Analysis Models

Fig. 3 shows the structural analysis model.

The RC slab was modelled using the shell element, and both the unprotected secondary beam and the primary beam was modelled using the beam element. The vertical-direction displacement at the floor edge was to be restricted, and a vertical load of 11.2 kN/m^2 was uniformly given to the entire surface of the composite floor.

The stress-strain relationship of concrete and steel at high temperature followed those prescribed in Eurocode 4[2)], and the strength of structural materials at room temperature was set based on the material test results. Specifically, the compression strength of the concrete was set at 36.1 N/mm^2, and its tensile strength at 0 N/mm^2; and the yield point of the reinforcing bars was set at 380 N/mm^2, and the yield point of the secondary beam and primary beam respectively at 349 N/mm^2 and 298 N/mm^2 (refer to Tables 1 and 2).

Fig. 2 Comparison of Temperature Hysteresis inside RC Slab

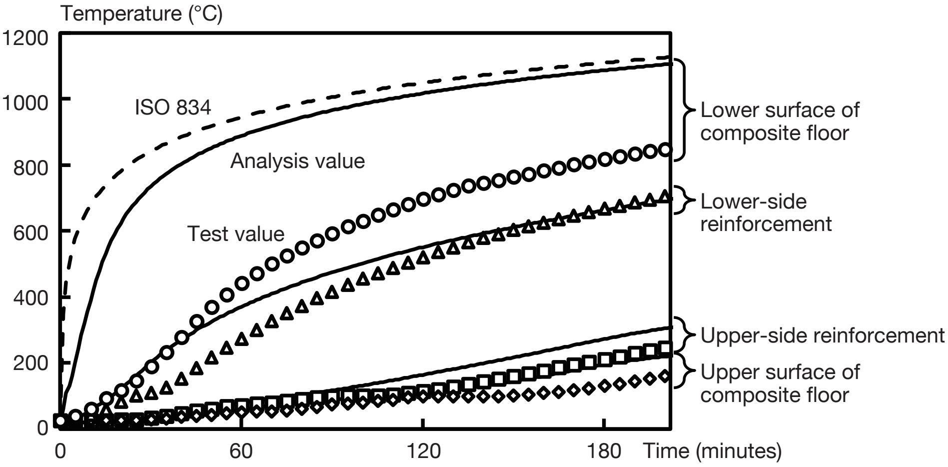


Fig. 3 Structural Analysis Model

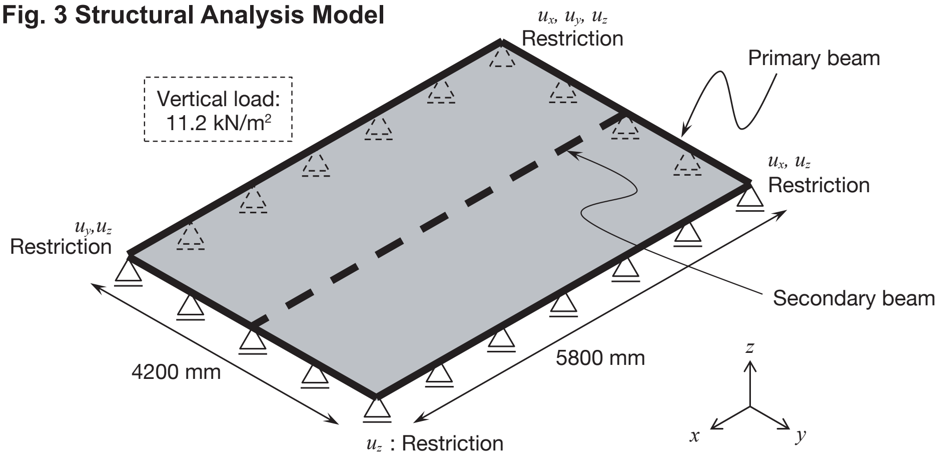


Table 1 Material Properties of Steel

Material	Young's modulus (N/mm^2)	Yield strength (N/mm^2)	Poisson ratio
SN400B	210,000	349	0.3
SS400	210,000	298	0.3
SD295A	210,000	380	0.3

Table 2 Material Properties of Concrete

Material	Compressive strength (N/mm^2)	Tensile strength (N/mm^2)	Poisson ratio
Fc24	36.1	0	0.2

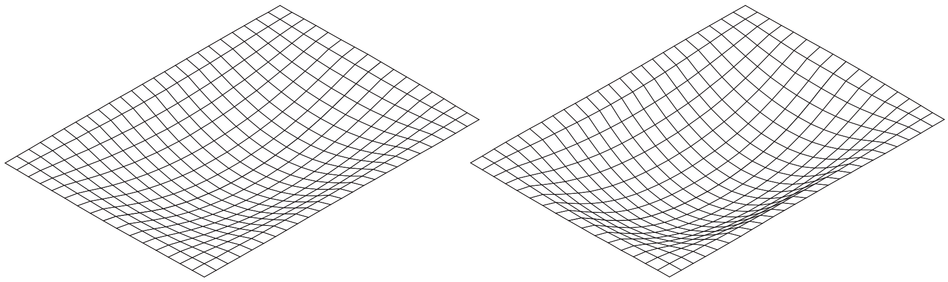
• Results of Analysis

Fig. 4 shows the floor deformation (magnification: 5 times) as an example of structural analysis. It can be seen in the figure that with the lapse of heating duration, the deflection of the composite floor increases.

Fig. 5 shows the deflection hysteresis at the center of the composite floor. It was understood from the figure that, while the analysis value is smaller than the test value up to heating for about 45 minutes, the trend of the increase in deflection after 45 minutes of heating was nearly identical for the analysis and test values. The deflection at the time of 132nd minute (stop heating) was 191 mm in the test and 197 mm in the analysis, which showed favorable consistency between them.

Fig. 6 shows the distribution of main stresses occurring within the composite floor surface by dividing the stresses into compression and tension. From the figure, while it was confirmed that the floor long side-direction tension stress due to the thermal expansion of the unprotected beam occurred after 5 minutes of heating, it can be understood that the compression stress occurred to the circumferential direction in the peripheral area of the composite floor after 30 minutes of heating, and at the same time a tension sphere was formed in the center of the composite floor. Further, with the lapse of heating duration, the temperature of lower-side reinforcement rose to cause the reduction of the strength of the composite floor, and as a result, the tension sphere expanded.

Fig. 4 Floor Deformation (Magnification: 5 Times)



(a) After 60 minutes of heating (b) After 120 minutes of heating

Fig. 5 Comparison of Deflection Hysteresis in the Center of Composite Floor

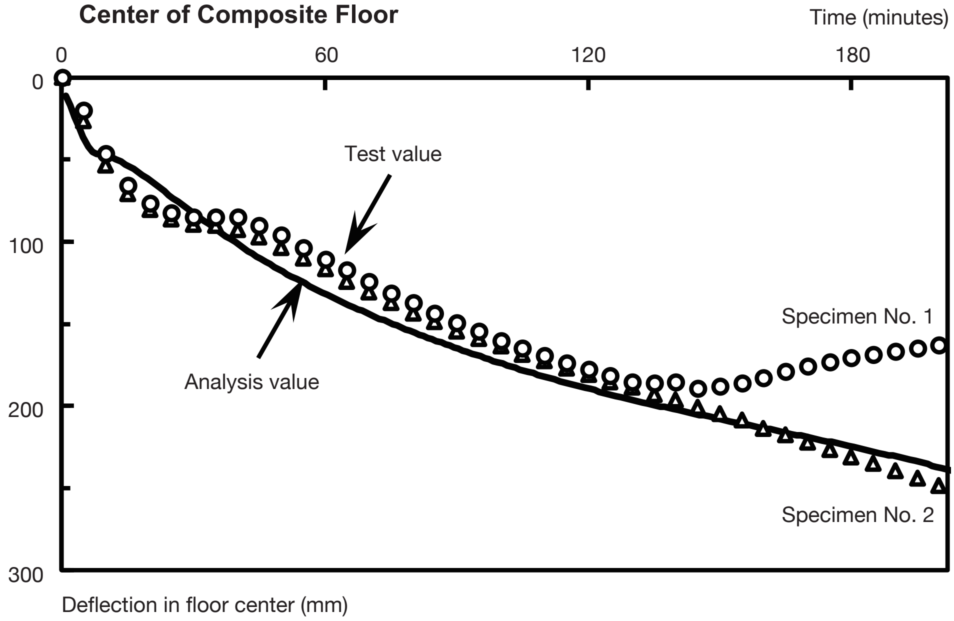
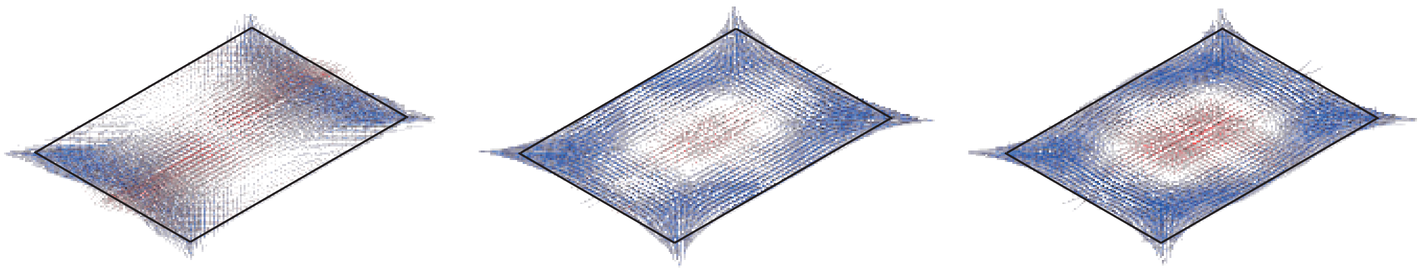


Fig. 6 Distribution of Main Stresses within Composite Floor Surface (Blue: Compression; Red: Tension)



(a) After 5 minutes of heating (b) After 30 minutes of heating (c) After 60 minutes of heating

Fig. 7 shows the transition of the temperature and axial force of the unprotected secondary beam. Incidentally, the tensile force was treated as a positive value. It is understood from the figure that the temperature of the unprotected secondary beam reached 450°C after 7 minutes of heating and that the compression axial force reached the maximum level at 930 kN. Then, as the temperature of the secondary beam increased, the axial force approached the level of 0.

It is forecast from these analysis results that in specimen No. 1 the vertical load can be continually supported due to membrane action.

♦♦♦♦♦

In Part 3, examinations by means of numerical analysis were conducted targeting the load-bearing fire test for full-scale RC slabs to confirm the accuracy of thermal and structural analysis. Further, the stress distribution within the composite floor surface when the membrane action works was shown.

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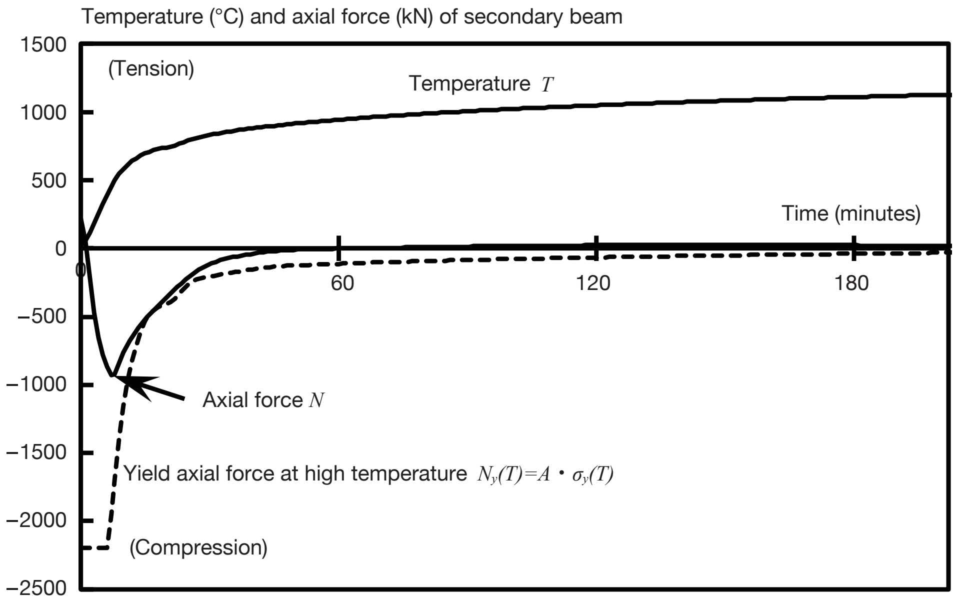
1) Gerna, T., Franssen, J.M.: Modeling structures in fire with SAFIR®: Theoretical background and capabilities, Journal of Structural Fire Engineering, Vol. 8, issue 3, pp. 300-323, 2017.
2) Eurocode 4: Design of composite steel and concrete structures, Part 1.2: General rules - Structural fire design, EN 1994-1-2 (2005), 2005.

Fig. 7 Transition of Temperature and Axial Force of Unprotected Secondary Beam



Membrane Actions of Reinforced Concrete Floor Slabs Exposed to Fire

by Fuminobu Ozaki, Associate Professor, Nagoya University

Fuminobu Ozaki: Graduating from the School of Architecture and Building Engineering, Tsukuba University, he finished the doctoral course at the School of Engineering, Tsukuba University and obtained Dr. Eng. in 2003. He assumed his current position as Associate Professor, the Graduate School of Environmental Studies, Nagoya University in 2013.

Part 4: Fire Resistant Design of Steel Structures in Japan and Future Development

Fire Resistant Design of Steel Structures in Japan

It is the *AIJ Recommendations for Fire Resistant Design of Steel Structures*, prepared by the Architectural Institute of Japan (AIJ), that is adopted as common guidelines for fire resistant design methods for steel structures in Japan.

Following the revision of the Building Standard Law of Japan in 2000, two design methods were introduced for the fire resistant design of buildings: design based on specification route (Route A) and design based on performance route (Routes B and C). With the introduction of Routes B and C, it has legally become possible to apply performance-based fire resistant designs for steel structures.

In the verification of the fire resistance of steel structures by means of simple calculations in Route B, the basic concept stated in the *AIJ Recommendations* has been applied, and the basic concept has also been applied in the verification of the fire resistance of steel structures that requires advanced calculations in Route C. To that end, the *AIJ Recommendations* has been accepted as the guidelines that play an important role in the practical fire resistant design of steel structures in Japan.

AIJ Recommendations for Fire Resistant Design of Steel Structures

The first edition of the *AIJ Recommendations* was published in 1999, and the third edition was published in 2017. In the latest 2017 edition, a new fire resistance assessment method was proposed and the existing assessment method was improved by incorporating into the *AIJ Recommendations* many attainments of the latest research on the fire resistance of steel structures. Noting the ultimate strength of steel framing, the *AIJ Recommendations* provides fire-resistant design methods for overall and partial steel framing in an event of fire. Fig. 1 shows the procedure of fire resistant design shown in the *AIJ Recommendations*.

A notable feature of the *AIJ Recommendations* lies in that the fire resistant design of steel structures is able to be completed using just the *AIJ Recommendations*. The left side in the figure shows the design route that assesses fire actions, and the right side shows the route that assesses structural strength. As a result, the simple fire resistant design method can be put into practical use due to the complete division of fire resistant design elements into these two routes as shown in the figure, and further examination by means of advanced numerical calculation is not entirely required.

The *AIJ Recommendations* is prepared based on existing research in which the thermal stress during fires does not in most cases affect the ultimate strength of steel structures, which thus allows for the realization of performance assessment based on a comparison between the maximum steel member temperature owing to the fire action and the collapse temperature when the steel member collapses (see Fig. 1).

In Japan, a certain level of or higher member strength and plastic deformation capacity are imparted with steel structures through seismic design, and in such steel structures, because high-temperature strength and plastic deformation capacity can be expected to demonstrate during fire, the effect of thermal stress on steel structures at the ultimate state has become extremely small. Meanwhile, regarding the flexural or local buckling of columns and the high-temperature strength of high-strength bolt connections and other structural members insufficient in high-temperature strength or plastic deformation capacity during fires, the collapse temperature is reduced by providing specific assessment methods for each.

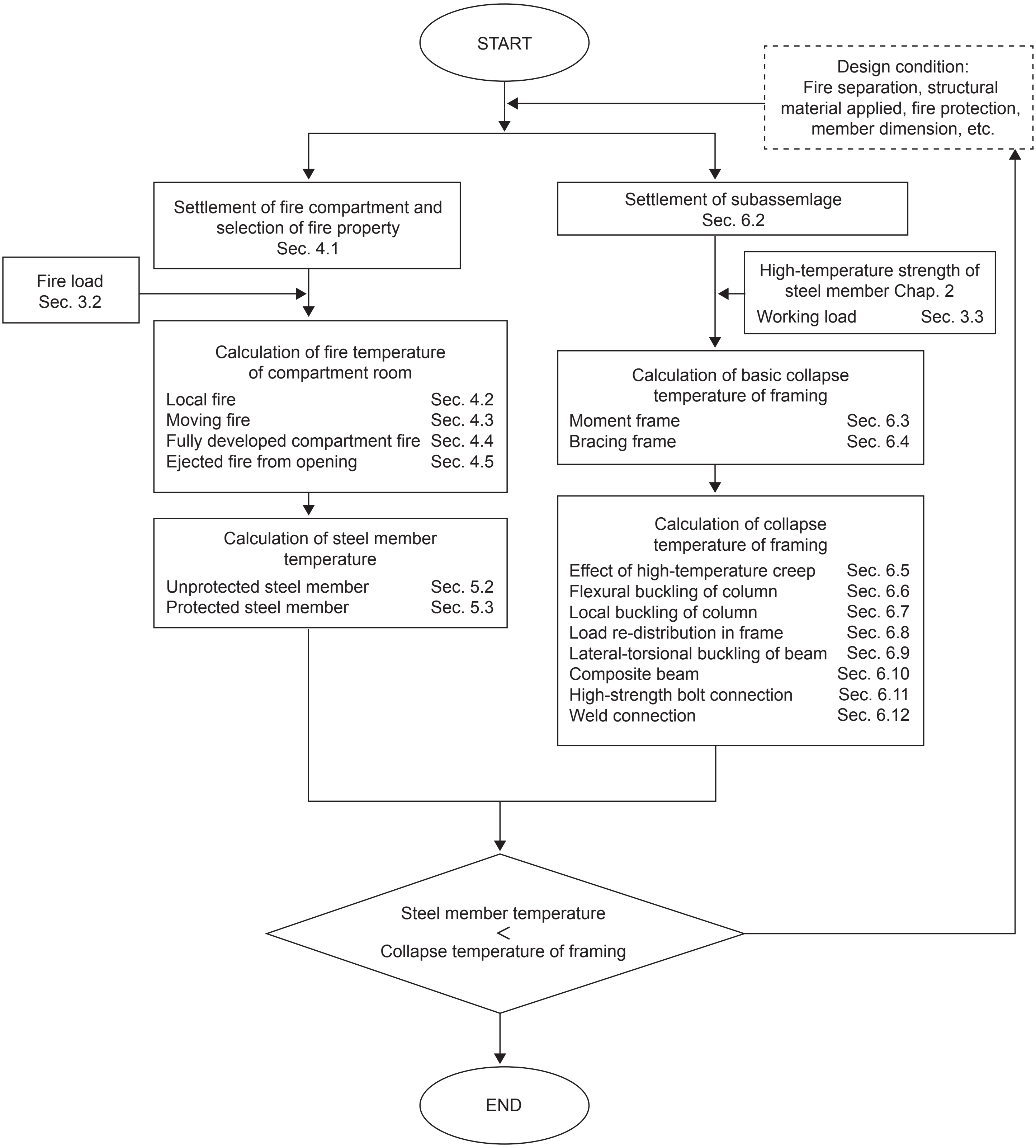
In the third edition, a design method was provided that prevents the entire collapse of a building during fire by assessing the load re-distribution capacity (redundancy) of the overall framing, and it has thus become possible to improve the collapse temperature of framings through the optimization of redundancy.

Future Development of Fire Resistant Design

Steady efforts are being directed towards the publication of the fourth edition of the *AIJ Recommendations*, into which plans call for the incorporation of research results on the reinforced-concrete flooring system introduced in the preceding Parts 1~3. Further, the scope of steel grades subjected to fire resistant design is being expanded and it is examined to incorporate high-strength steel products in the future edition of the *AIJ Recommendations*.

On the other hand, in settling the steel effective strength for use in fire resistant design, the traditional approach has been adopted since the first edition. Specifically, the design strength has been settled at zero at fire temperatures of 750°C or higher, and therefore fire resistant design has been required to be made so that the collapse temperature becomes

Fig. 1 Fire Resistant Design Procedure in *AIJ Recommendations for Fire Resistant Design of Steel Structures*



750°C or lower. At the time of publication of the first edition of the *AIJ Recommendations*, because of the lack of available test data for steel effective strength, a comparatively large safety factor was provided for steel effective strength. But relevant test data has steadily been accumulated since, and reliable theories have also been developed for the fire resistant design of steel structures. As a result, it is considered that it will be possible in the future to introduce limit state fire resistant designs represented by the LRFD that will be carried out without counting on the excess safety factor.

When the concerns involved in an upper-limit design temperature of 750°C or higher are solved and the results of research on reinforced-concrete flooring systems introduced in Parts 1~3 are reflected in future editions of the *AIJ Recommendations*, building design employing unprotected steel structural members (members with no fire protection covering) will become easier. To that end, the attractiveness of steel structures is expected to further improve.

Mikiyama Park Gymnasium

—Sharply-designed Steel Truss Framing Lightly Floats Up into a Large Arena Space—

Ishimoto Architectural & Engineering Firm, Inc.
Nippon Steel Engineering Co., Ltd.

A notable feature of Miki City in Hyogo Prefecture is the hardware industry that has experienced sustained development since old times, and currently the city is known as a hardware town that produces carpenter tools that are ranked as the best in Japan.

In the project of the Mikiyama Park Gymnasium, we intended that this arena will be loved by Miki citizens by expressing "Hardware Town Miki." A remarkable feature is that the truss frame with small curves gives the inner space an image of sharp hardware (Photo 1).

Photo 1 Finely-designed space truss framing

"Hardware Eagle," a Symbol of Miki City

The "hardware eagle" is an eagle-shaped art object assembled employing a total of 3,329 saws, kitchen knives and other hardware products, all of which are Miki-made hardware products (see Photo 2). The art object with a total wing length extending to 5 m is manually assembled, consuming as much as 8 hours, whenever the Miki hardware festival is held every year. It is a precious symbol for Miki citizens.

Photo 2 Hardware eagle, a symbol of the hardware town of Miki

Image of Gymnasium Arena Space

At the initial stage of design of the Mikiyama Park Gymnasium, the architect involved in the project had an image of an arena space that is covered with tree-state framings and surrounded with a grove of trees growing in the forest.

As the design proceeded, the annual Miki hardware festival was held. When the design team joined the Miki hardware festival, the team was fascinated by the power and beauty of the hardware eagle. Since then, the team was eager to pursue and express an image of hardware rather than tree-like structure.

In a frame composed of a lot of structural members, the impression and atmosphere are determined by the detail of joints and the shape of steel members. In order to save the construction cost, we aimed to achieve an elegant structure using usual and low-price steel members. Then we added an idea of applying a hardware image to the detail design and developed an idea that strengthens the beauty of the edge and surface of truss framing.

Building Plan

The newly-built gymnasium was connected to an existing swimming pool building via a passage to form an integrated facility. The building plan called for the arrangement of an arena and exercise rooms on the first floor and the arrangement of spectator seating and terraces on the second floor, and at the same time the linkage of an entire facility using a *passage*s-like corridor that runs through the entire facility from south to north. (Refer to Figs. 1 and 2)

A variety of exercises can be taken at the gymnasium. In this connection, some devices to promote motivation for exercising were provided. For example, windows were arranged in various places of the corridor so that visitors can see enjoying exercises of others too.

Fig. 1 1st-floor Floor Plan

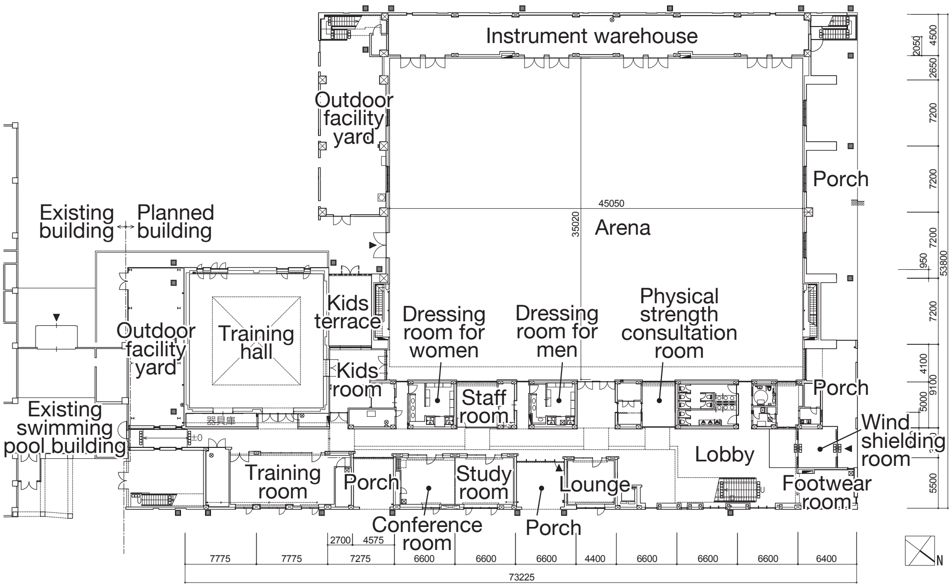
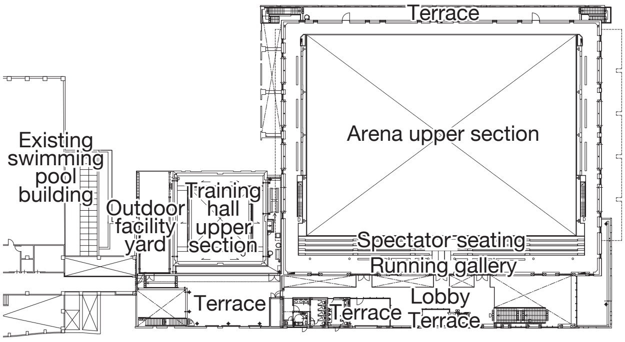


Fig. 2 2nd-floor Floor Plan



Outline of Structural Plan

For the main structure of the gymnasium, a reinforced-concrete structure was adopted for the lower structure, and a steel structure for the roof. The arch truss base section of the roof was settled at the level of the second-floor spectator seating, and the base was joined with the seat floor using pivotal bearings. (Refer to Figs. 3 and 4, Photo 3)

Outline of Arch Trusses

An arch truss configuration was formed employing the lower chord member of the three circles-triangles type and the upper chord member of the circle type similar to that of the lower chord member. H-beams were applied for the upper chord member, built-up steel angles were applied for the lower chord member, and covering plates were added to the lower chord nearby the base subjected to large axial force, thereby structuring the arch truss with a triangular cross section.

As the inner space with "hardware town MIKI"-specific structure, the design team tried to design structural details that express not only the clear and straight edge but the shining surface of sharpened steel.

• Expression of "Refined Lines" of Hardware

We took the lower chord member as the main item that imparts deep impression with the arena. The bottom edge of lower chord member curves from the lower level through the top so that visitors can see the elegant curved line floating in the arena. T-shaped steel members with two steel angles, used as the lattice member, make the edge clear. The gap between two angles seems like the line that divides the flat surface.

Photo 3 Full view of Mikiyama Park Gymnasium

Fig. 3 Cross Section

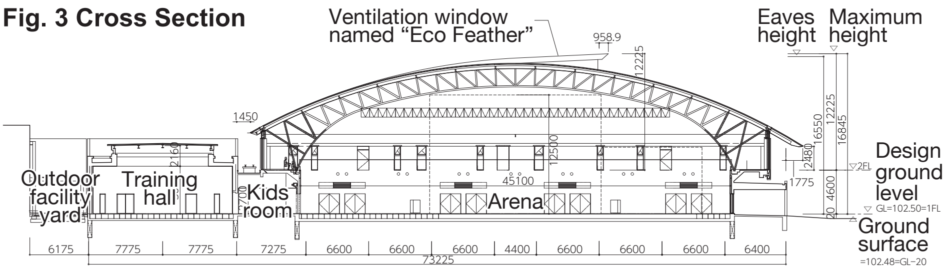


Fig. 4 Outline of Structural Plan

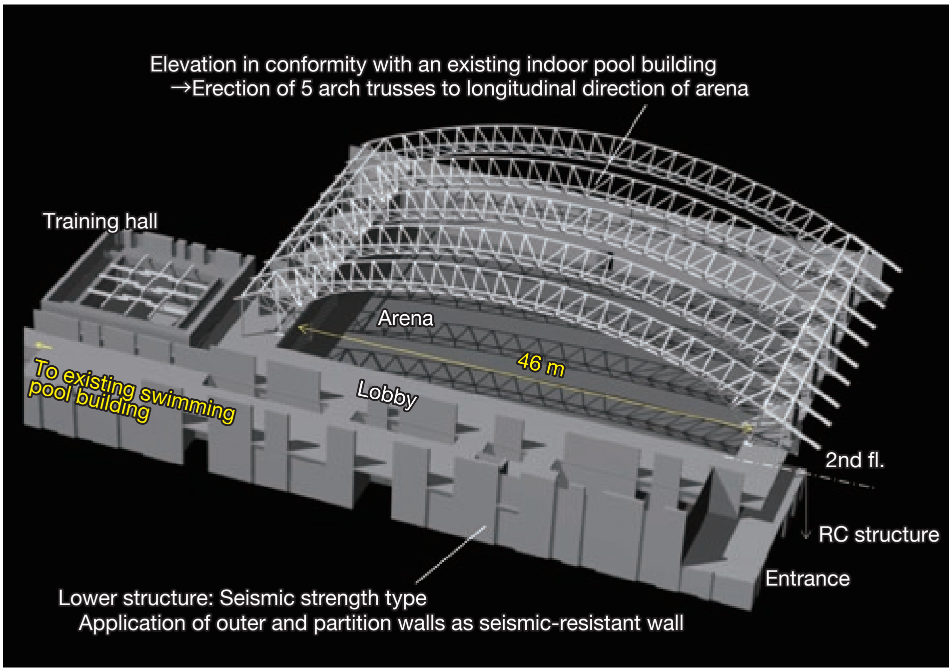


Fig. 5 (1) Configuration of Truss

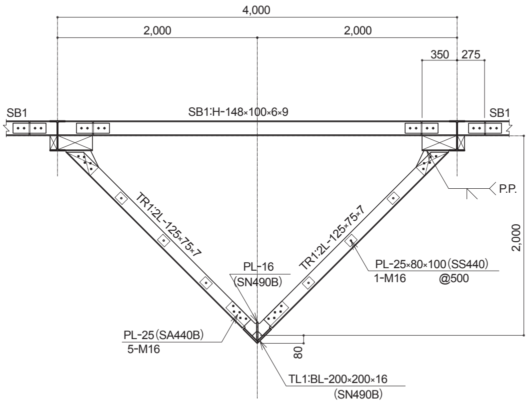
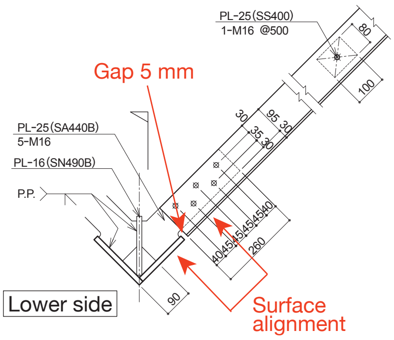


Fig. 5 (2) Detail of Truss



• Expression of "Fine Surfaces and Brilliance" of Hardware

We created the details in which the lower chord member and the lattice member are aligned to realize an identical structural plane (Fig. 5).

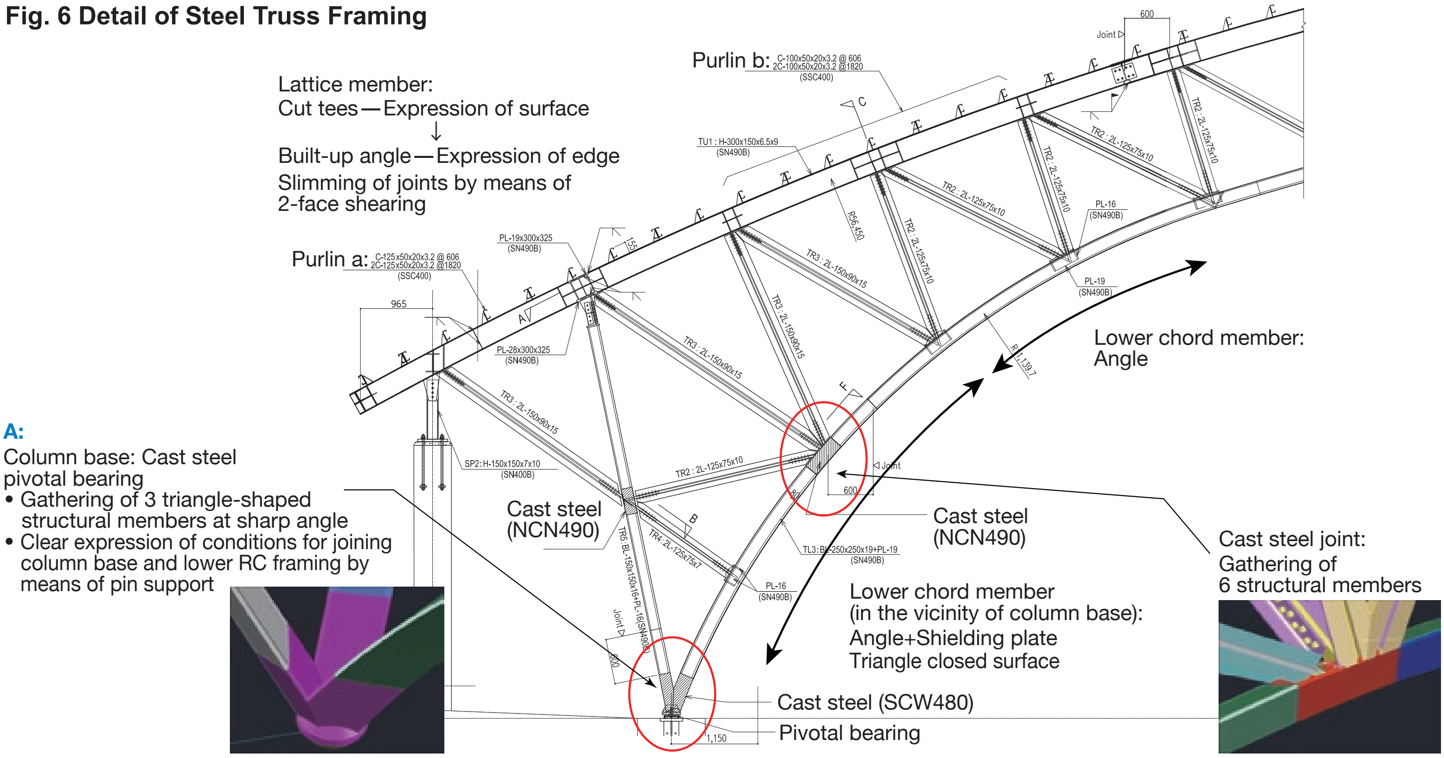
Fig. 6 shows the details of the truss framing. The structural members gather at many joints in various angles. Therefore, we verified the detail in terms of the appearance from various viewpoints and the construction efficiency using mock-ups and 3D simulation.

For the column base (A in the figure), cast steel was applied. The dimension of the steel-frame members adopted were H-beam—H-300×150×6.5×9 (SN490B) for the upper chord member; built-up angle—BL-250×250×19 + PL-19 (SN490B) for the lower chord member and angle—2L-125×75×7 for the lattice member.

Steel-frame Fabrication

The design team and the steel-frame fabrication team discussed repeatedly to share design concepts and intentions. During this process, the steel-frame fabrication team steadily understood that how to attractively assemble the framing was an important factor in creating an attractive arena space and how to secure high finishing accuracy was a demanding element for the interior. To meet these requirements and as a member of one team of the project, the steel-frame fabrication team resolved to make their work unique to the hardware town.

Fig. 6 Detail of Steel Truss Framing



Many skillful engineers joined the team to find out the best solution. We thought this timing as the most important milestone and raised two keynotes:

One was to show the "clear edge of steel members." The edge of the lower chord member was required to go straight from the side to side like the Japanese sword.

Another was to align the surface of structural members applied. Each surface of lower chord and lattice members was required to be aligned in one surface. The clear finish of aligned surface depends on surface finish of welds. We chose to sharpen the welding finish although it required high cost and time consuming process.

Tasks and Responses in Steel-frame Fabrication

In cases when we adopt the angle and it is bent as the lower chord member, the essential member, there was a possibility that a large strain would occur at the end of flange, To solve this problem, the team drew unfolded drawings of the angles by using 3D-CAD. The fan shape was chosen as the shape of unfolded angles. Two fan-shaped plates were weld-joined to manufacture the built-up angle with a curved configuration. (Refer to Photo 4 and Fig. 7)

On-site Erection of Steel Framing

In order to secure the erection accuracy of the lower chord members, the construction team planned to build the truss by single unit instead of assembling units on the ground so that truss assembly could be done only with slight adjustments. Assembly work was managed not to cause the errors during building of truss framing.

The on-site welding of lower chord members was unusual in terms of welding-line angles and welding directions. The team built mockups in order to confirm the appearance and quality, and decided to grind the welding line aligned with each other (Photo 5). For high-strength bolt joint of the chord and lattice members, how to insert each bolt was planned in advance considering bolt-tightening efficiency and appearance from the spectator seating (Photo 6).

(Continued overleaf)

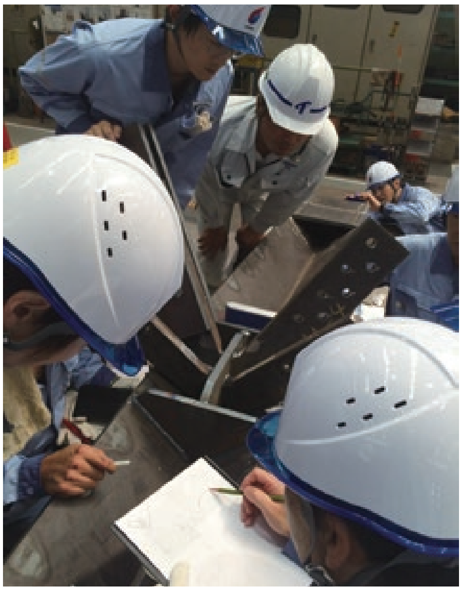
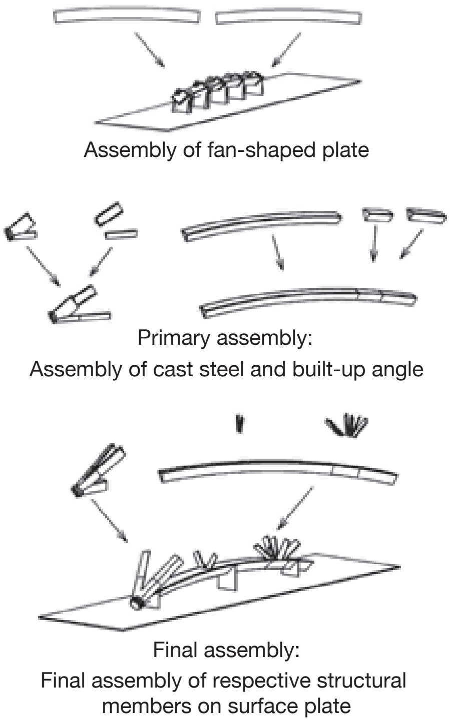


Photo 5-1 Confirmation of the configuration of joints using mockups

Photo 5-2 Checking of angle grinding condition to make the angle edge conspicuous

Photo 5-3 Examination of the configuration of the column base cast steel using a Styrofoam mockup

Fig. 7 Fabrication Method for Structural Members



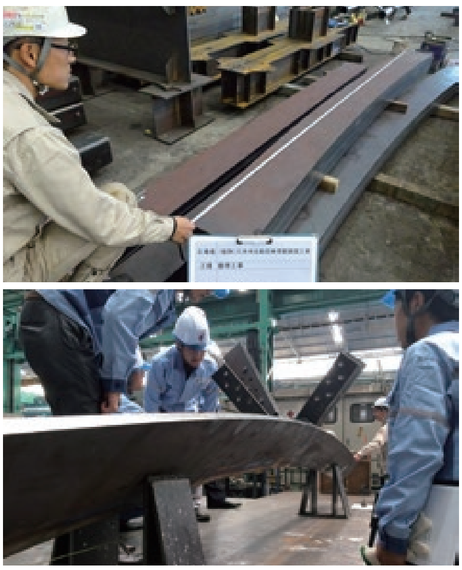


Photo 4 The fan-shape plate for manufacturing the built-up angle and confirmation of the configuration of the lower chord member using mockups

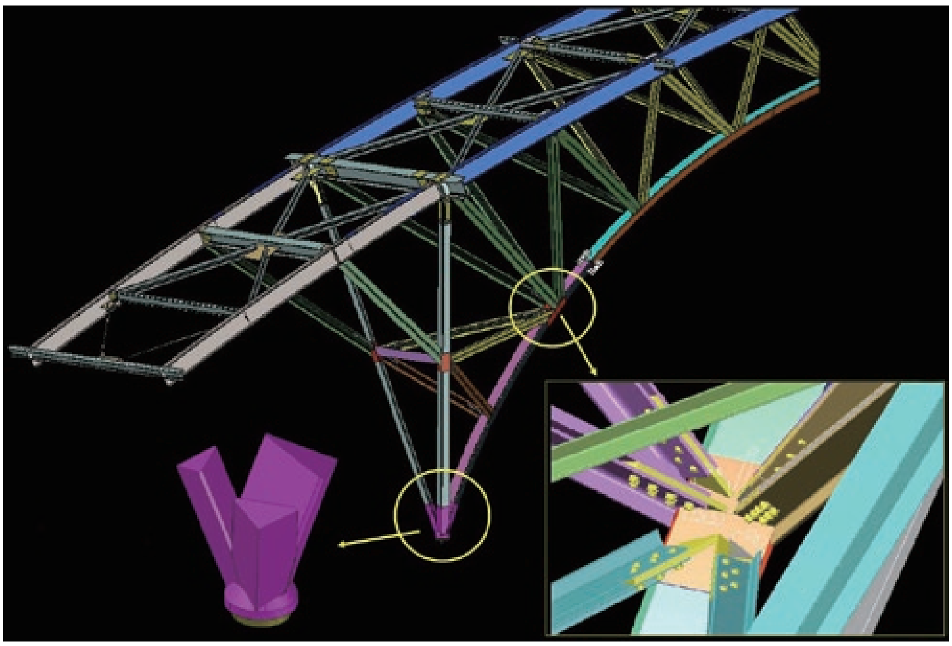


Photo 6 Checking of bolting work by means of 3D-CAD

Before entering into the jacking down phase, the team simulated the procedure and analyzed the degree of displacement. Jacking up was carried out in 2 steps. The result was almost same as pre-construction analysis.

♦♦♦♦♦

Construction of the Mikiyama Park Gymnasium was successfully completed, and when looking up at the completed arena of the gymnasium, the space truss framing structured with great cares stands in overwhelming beauty. When we see the great completion of the Mikiyama Park Gymnasium, the difficult time that we had has gone away. And we'd like to share our feeling with Miki hardware craftsmen and to say "We did IT."

Fine Arena Space Worthy of the Hardware Town of Miki

In previous architectural designs, steel pipes have commonly been used for the space trusses. In this project, the team pursued "Miki-specific structure" and reached the unique structure that featured "LINE" and "SURFACE" by the use of steel angles. (See Photo 7)

The truss frame "flying" in the arena is painted carefully and these sharpened edges remind the hardware and carpenter tools. We believe that we have achieved "Miki-specific space" with this unique structure detail.

Outline of Mikiyama **Park Gymnasium**

Location: Miki City, Hyogo Prefecture
Project owner: Miki City Office
Main application: Gymnasium
Area
- Site area: 239,922.00 m^2
- Building area: 3,538.33 m^2
- Total floor area: 4,203.47 m^2

Structure
- Aboveground: RC structure, partly steel structure
- Gymnasium roof: One-way steel-frame truss
- Pile, foundation: Spread foundation (partly shallow-layer improvement)

No. of stories: 2 aboveground
Maximum height: 16.84 m
Architectural design: Ishimoto Architectural & Engineering Firm, Inc.
Structural design: Ishimoto Architectural & Engineering Firm, Inc.
Construction
- Building: Takashina Corporation
- Gymnasium roof: Nippon Steel Engineering Co., Ltd.

Design period: June 2015~March 2016
Construction period: June 2016~July 2017

Photo: Koichi Torimura
(Part of photos: Ishimoto Architectural & Engineering Firm, Inc., Nippon Steel Engineering Co., Ltd.)

Photo 7 New space truss structure stressing the "structural 'LINE and SURFACE' of framing" with the use of steel angles

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3-2-10, Nihonbashi Kayabacho, Chuo-ku, Tokyo 103-0025, Japan
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URL https://www.jisf.or.jp/en/index.html

Japanese Society of Steel Construction
3F Aminosan Kaikan Building, 3-15-8 Nihonbashi, Chuo-ku, Tokyo 103-0027, Japan
Phone: 81-3-3516-2151 Fax: 81-3-3516-2152
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