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【 t a i 】

"耐 (*tai*)" in Japanese, or withstanding, resistance in English

This issue, No. 40, features the fire resistance of steel structures and the advanced fire protection and fire-resistant design applied to these structures in Japan; it also discusses an improved approach to the on-site welding of steel structures.

In this issue.....

Special Feature: Fire-resistant Technology

1 Buildings and Fire Resistance

4 Fire Protection for Steel Structures

6 Fire-resistance Redundancy in Steel Framing

9 Surveys of Fire Damage of Steel Structures

12 Fire-resistant Design of TOKYO SKYTREE

15 Fire-resistant Steel

Steel Application Technology

17 On-site Welding of Steel Structures

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Buildings and Fire Resistance

—Development of Fire-resistant Design in Japan—

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Reasons to Impart Fire Resistance to Buildings

The frequency of building fires is naturally not very high. And, most buildings never experience even one fire during their service life. However, even while the possibility of a fire breaking out is low, once one does occur, it can cause seriously harmful effects to not only the building interior but the surrounding area as well. In order to keep such effects within an allowable range, it is necessary for buildings to provide specified fire resistance.

Currently, buildings must demonstrate the following capabilities during a fire:

(f1) Occupants (persons present in the building) can safely evacuate the building.

(f2) If some occupants cannot evacuate the building themselves, firefighters will be able to search for and safely rescue them.

(f3) Even if all or part of a building were to collapse, the collapse would not harm surrounding buildings.

(f4) Heat radiating from openings or other structural parts of the building will not cause fires to break out in surrounding buildings.

(f5) Property incorporated within the building is protected from the fire.

In order to demonstrate these capabilities, diverse measures are taken — fire prevention, confining a fire to a specified area, and preventing the destruction or collapse of a building due to heat from a fire. Fire resistance is commonly divided into three performance categories: load-bearing capacity (R), insulation (I) and integrity (E). In order to confine a fire to a specified area, insulation and integrity are important factors, and to prevent the destruction or collapse of a building, load-bearing capacity plays an important role. For example, in order to safely evacuate a building's occupants, the structural stability of the building, including evacuation routes, should be secured until the evacuation is finished. Further, in order to restrict a fire from spreading beyond the fire compartment, insulation and integrity are required for the walls and floors that serve as the boundaries of the fire compartment.

Development of Fire-resistant Design in Japan

The structural design work that imparts the required fire resistance to a building is called fire-resistant design. This work is conducted so that the aforementioned three types of performance are appropriately secured for the respective structural sections of a building. In Japan, the Building Standard Law was enacted in 1950 after the end of the World War II and, with other building regulations, extensively treats how to promote fire-resistant design. Several revisions have been made to the Building Standard Law, and the provisions of the law related to fire resistance prior to the revision in 2000 differ greatly from those after revision.

According to the Building Standard Law, in cases when capabilities (f1) through (f5) above cannot be demonstrated due to a fire and the resulting effect is severe, the building should be fire-resistant taking into account the use, scale and location of the building. Prior to the 2000 revision, the only available measure was to specify that the main structural members, such as the columns, beams, floors and walls, were composed of fire-resistant members employing incombustible materials. More specifically, pertaining to load-bearing capacity, it was required that the main structural members be composed of fire-resistant members and that these members must have the fire resistance times prescribed in Table 1 for the various main structural parts (in addition, it was required for floors, walls and roofs to have appropriate insulation and integrity). This approach is called design by compliance to prescriptive provisions.

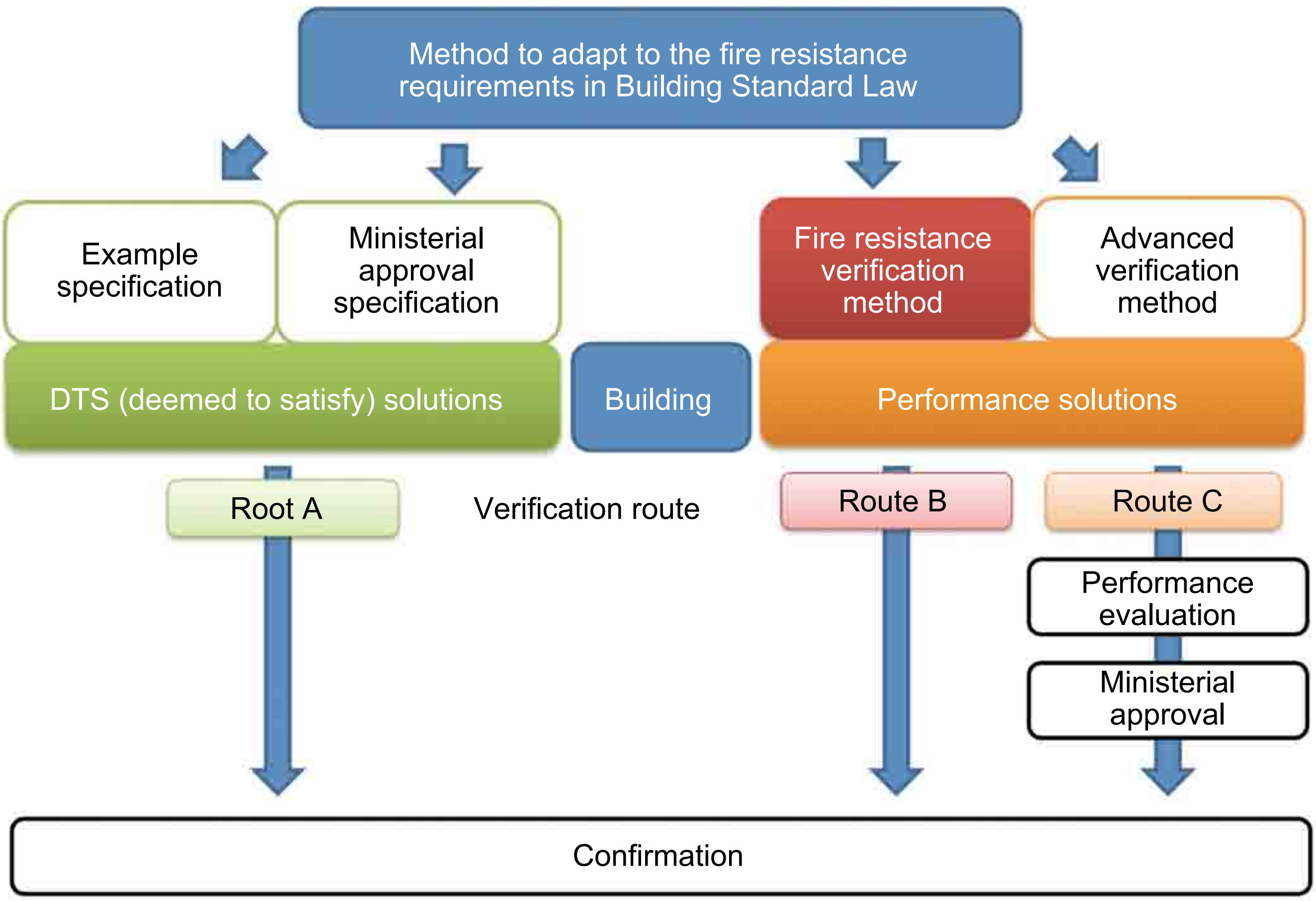
After its revision in 2000, the Building Standard Law incorporated performance-based design as well. Specifically, it prescribes that, in an assumed fire, a fire-resistant building structure be composed of main structural parts for which the fire resistance is confirmed to last until the fire is extinguished. As a means to ensure confirmation, the Law prescribes a fire resistance verification method.

Meanwhile, in future cases where new fire resistance technologies and verification methods have been developed and put into application but whose fire resistance cannot be confirmed using the calculation procedure prescribed in the Law, it is now possible for the Minister of Land, Infrastructure, Transport and Tourism to approve their adaptability based on the results of fire resistance

Table 1 Required Fire Resistance Time for Load-bearing Capacity

Parts of buildings \ Stories of buildings		Uppermost story, and second to fourth stories from the uppermost story	Fifth to fourteenth stories from the uppermost story	Fifteenth story or more from the uppermost story
Walls	Partition walls (Load bearing)	1 hour	2 hours	2 hours
	Exterior walls (Load bearing)	1 hour	2 hours	2 hours
Columns		1 hour	2 hours	3 hours
Floors		1 hour	2 hours	2 hours
Beams		1 hour	2 hours	3 hours
Roofs		30 minutes		
Stairs		30 minutes		

Fig. 1 Fire Resistance and Building Standard Law



evaluation conducted by an examination committee composed of experts.

Before 2000 only design by compliance to prescriptive provisions could be applied, except for limited exceptions. Currently, it is possible to apply performance-based design in addition to design by compliance to prescriptive provisions. (Refer to Fig. 1)

Example of a Large-scale Building Collapse Caused by Insufficient Fire-resistant Design

Even in apparently fine buildings, if the fire-resistant design is not adequately executed, the consequences for the building can be catastrophic. An example of adopting insufficient fire-resistant design which we investigated is introduced below.

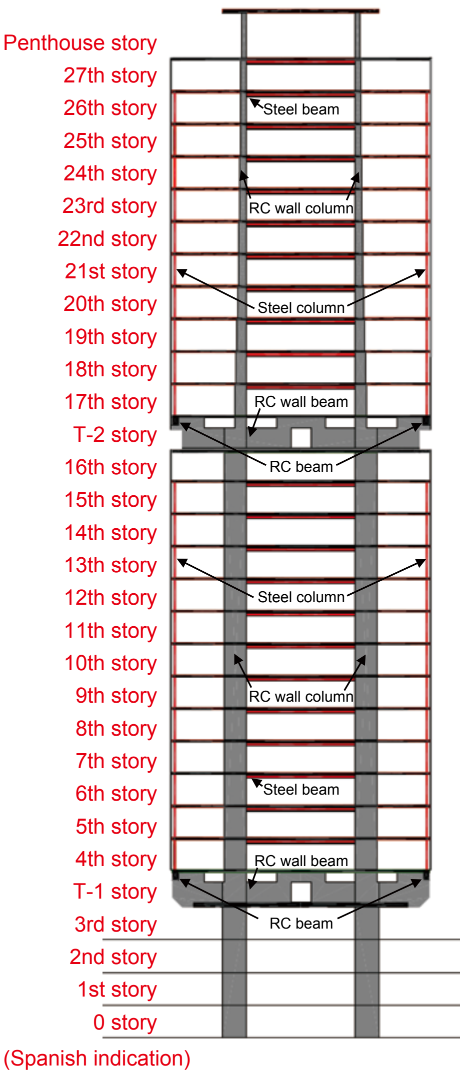
A fire broke out at midnight on February 12, 2005 (local time) in a 32-story high-rise building (Winsor Building) constructed in the AZCA area of downtown Madrid, the capitol of Spain. The fire rapidly spread both upwards and downwards, engulfing nearly every floor. A large-scale framing collapse occurred in the middle and upper stories that resulted in great amounts of curtain walls, framing members and building containments being scattered around the area surrounding the building. The fire was not only disastrous for the Winsor Building, but it also seriously affected the ability of the Spanish capitol to function by causing both the closure of roads in the area surrounding the business center and the suspension of subway operations.

Photo 1 shows an overall view of the damage in the latter part of March, one and a half months after the fire. Most of the building's columns, beams, floors and bearing walls were composed of reinforced-concrete structures, while the columns of the building's outer periphery, the end sections of the wide open office spaces, were made of steel. With two technical floors located in the middle and lower stories, the building was structurally divided into a low-rise section (up to the third floor above ground), a medium-rise section (fourth to sixteenth floors) and a high-rise section (seventeenth floor and higher). Fig. 2 shows the framing elevation, and Fig. 3 the floor plan of the seventeenth to twenty-sixth floors, including the twenty-first floor where the fire broke out.

The Winsor Building was completed in 1976 in conformance with the fire resistance standards of the day. However, as shown in the photo, the outer periphery of the high-rise section together with the floor slabs caused a large-scale collapse, and at the same time

Photo 1 Fire damage of Winsor Building, one and a half months after the fire

Fig. 2 Framing Elevation of Winsor Building



the medium-rise section that sandwiched the technical floor was totally destroyed by the fire. In current fire-resistant design, even if a fire were to break out, it would be confined within a fire compartment, it would not spread to multiple stories and the columns, floors and other main structural sections would not easily cause a collapse.

Because the Winsor Building was designed based on the building regulation standards of the time, fire protection was not provided for the steel columns on the building's outer periphery. Further, the connections between the floors and the exterior wall panels, important factors in creating fire compartments between floors, adopted a structure that easily ignited and allowed the fire to penetrate beyond its area of origin (refer to Fig. 4). In this way, an appropriate fire-resistant design was not adopted for the building. Consequently, the fire spread throughout entire sections of the building, thereby leading to a large-scale collapse. Because the fire broke out at midnight on a Saturday, when only a few persons were in the building, it was fortunate that no one died.

Toward Further Enhancement of the Fire Resistance of Buildings

In this article, we briefly described the need to require fire resistance in building construction, the development of fire-resistant design in Japan and an example of a large-scale collapse of a high-rise building that was attributable to the adoption of an inappropriate fire-resistant design. In the fire-resistant design that currently prevails in Japan, the main goal of realizing buildings that are safe against fire has been attained. However, in order to improve the appropriateness of fire-resistant design and its application, R&D will have to take into account the following five tasks:

- Understanding the high-temperature properties of various steel products
- Development of fire protection that considers application efficiency, durability and environmental concerns
- Promotion of framing plans rich in fire resistance redundancy
- Development of a standardized testing method that can accurately confirm performance
- Structuring of social framework to assess the fire resistance of building as a whole (engineers and evaluators) ■

Fig. 3 Typical Plan (Upper Stories) of Winsor Building

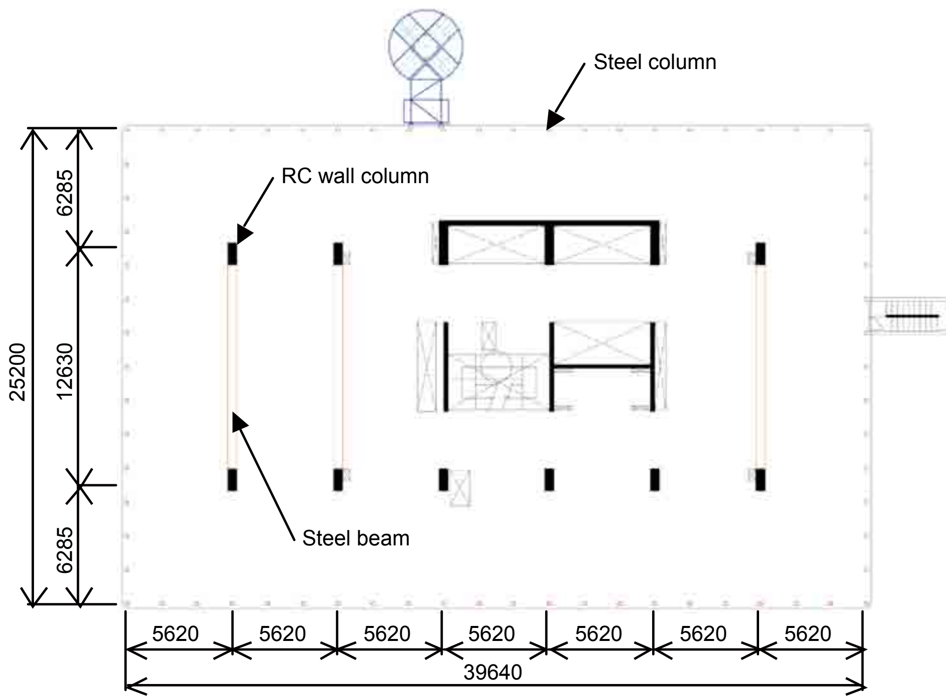
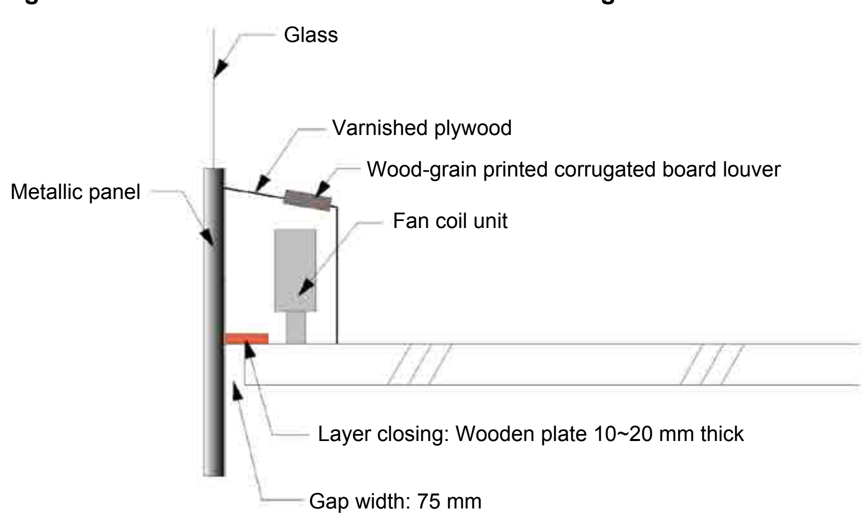


Fig. 4 Section of Perimeter Area of Winsor Building



Fire Protection for Steel Structures

by Shigeki Tasaka
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Outline of Fire Protection

Because fire resistance is not directly imparted to the columns and beams of steel structures, it is necessary to provide fire protection in order to protect a building from fire. Among the fire protections currently in use are spraying, coating, wrapping fire protection and formed boards, which are commonly applied in a single layer or a laminated manner.

However, with exterior wall sections, it is not possible to create distance between the exterior wall and the columns or beams, thereby making it impossible to apply fire protection to the contact points using only single-layered or laminated materials. In order to solve this problem, it is necessary to treat exterior wall members as part of the fire protection system for columns and beams and this has led to the development of composite methods of fire protection.

In Japan, when applying a specified fire protection method to a fire-resistant structure, the specified fire protection must satisfy the performance prescribed in the Building Standard Law. The specified fire protection method is tested and evaluated by a designated performance evaluation organization, and ministerial approval is given when the fire protection satisfies the required performance.

The fire protections that have obtained ministerial approval are publicly listed on the website of the Ministry of Land, Infrastructure, Transport and Tourism (MLIT). Table 1 shows the approval number. Of the total approvals, composite fire protection structures account for nearly half. Further, a total of 35 approvals are given for foam intumescent coatings applied to columns, and 19 to beams.

In Japan, sprayed rockwool and calcium silicate board are frequently applied in practical fire protection work, and sprayed rockwool and precast concrete board or ALC panels are frequently used in composite fire protection construction.

Type of Fire Protection Materials

Ministerial approval by MLIT is given to roughly two types of fire protection.

- Fire protection structures that use single-layer or laminated material to coat the four sides of column members and three sides (excluding the surface facing the floor) of beam members
- Columns—One surface is protected by the exterior wall and the remaining three surfaces with a single-layer or laminated material; Beams—One surface is protected by the exterior wall, and the remaining two surfaces by a single-layer or laminated material (excluding the surface facing the floor) (Refer to Figs. 1 and 2)

The latter type is classified as a composite fire protection structure, in which the exterior walls provide part of the fire protection. The connection of the exterior wall to the fire protection material is reinforced using a backing material, ribs or reinforcing members to eliminate any gaps.

Kind of Fire Protection Materials

Fire protection materials can roughly be classified into the following four types:

• Sprays and coatings

Rockwool, gypsum materials and cement are directly sprayed or coated on the steel product. These materials are commonly applied while wet. Photo 1 shows an application example of sprayed rockwool fire protection.

• Wraps

Rockwool felt, ceramic wool brackets and other inorganic fiber felts are wrapped around the steel product. These materials are commonly applied dry and are fastened using a fastening member. Photo 2 shows an application example of inorganic fiber felt fire pro-

Fig. 1 Example of Composite Fire Protection Structure of Steel Column

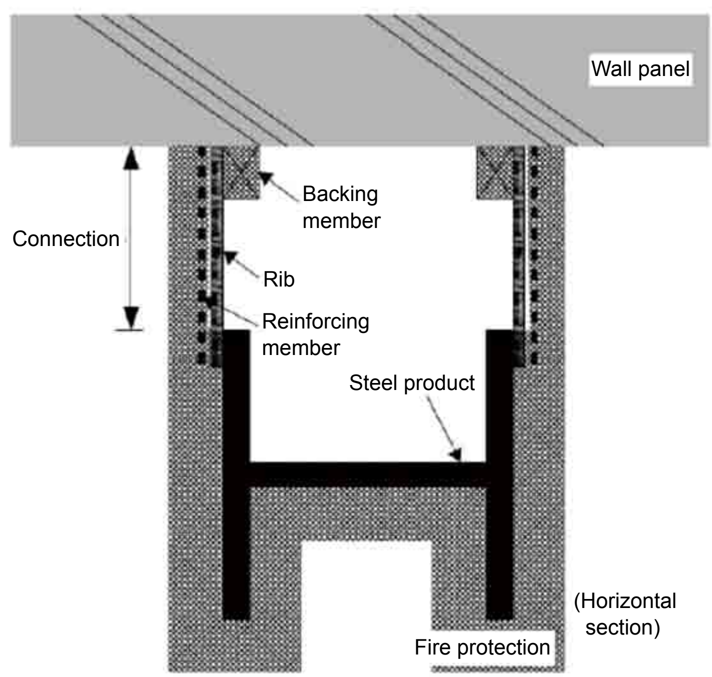
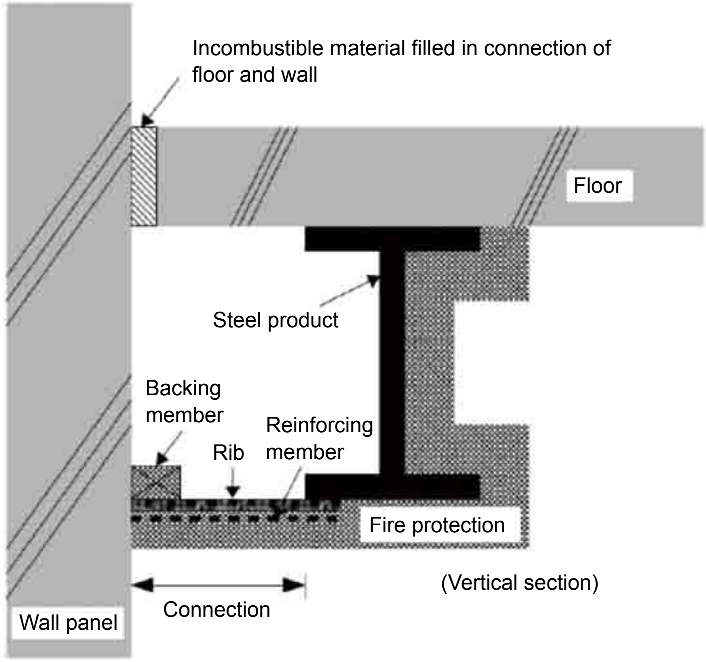


Table 1 Ministerial Approval of Fire Protection for Steel Structures

Structure	Required fire safety performance	No. of approval	Of which, approval for intumescent coating
Column	1 hour	450	19
	2 hours	313	16
	3 hours	229	0
Beam	1 hour	333	13
	2 hours	262	6
	3 hours	139	0

Fig. 2 Example of Composite Fire Protection Structure of Steel Beam



tection.

- **Formed materials**

Board members such as fiber-mixed calcium silicate board, gypsum board and wooden board are attached or pasted to the steel product. These materials are commonly applied dry and are fastened using a fastening member or an adhesive agent. Photo 3 shows an application example of fiber-mixed calcium silicate board.

- **Intumescent coatings**

The top coat and a base coat of foamed coating material are directly applied in a laminated state to the steel product. Photo 4 shows an application example of foamed intumescent coating.

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Among other fire protection materials are thermal-expansion sheet members and moisture aluminum packs. In addition, fire protection consisting of multiple materials is available. ■

Photo 1 Application example of spray rockwool fire protection

Photo 3 Application example of fiber-mixed calcium silicate board fire protection

Photo 2 Application example of inorganic fiber felt fire protection

Photo 4 Application example of foamed intumescent coating

Fire-resistance Redundancy in Steel Framing

by Kenichi Ikeda
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Concept of Redundancy in Steel Framing during a Fire

The assessment of framing redundancy during a fire differs from room-temperature assessments, and thus it is necessary to examine the external forces at work during a fire and the changes in strength characteristics of the structural members that these forces cause.

The first factor to be taken into account in an examination of the working external forces is that a fire breaks out in one part of a building. Fig. 1 shows the differences in the external forces dealt with in fire-resistant design and seismic design. The Earth's gravitational load works vertically throughout an entire building structure; and, during an earthquake, the ground vibrates and the resultant seismic energy also works not only on the foundation but on the entire building structure as well. On the other hand, because a fire generally breaks out in only part of a building and is extinguished before spreading through the entire building structure, temperature loads caused by fires work on only part of a building. Accordingly, the temperature of structural members located in the space where a fire breaks out will rise, but members located in sections untouched by the fire will not be affected.

Another factor to be taken into account with regard to the strength characteristics of structural members during fire is that the temperature of these members rises due to the heat, and as the temperature of a member rises, it not only loses strength and rigidity but experiences thermal expansion as well. Fig. 2 shows a concept rendering of the behaviors of building structures during a fire. The structural framing undergoes deformation due to the thermal expansion that occurs in the members in the early stages of the fire. While the length of columns normally approximates the height of the floor, the length of the beams used in steel framing is several times the column length. Naturally the thermal expansion of the beams is significant during a fire. The thermal expansion of the beams occurs in the axial direction, but the

Fig. 1 Difference of External Force between Fire-resistant Design and Seismic Design

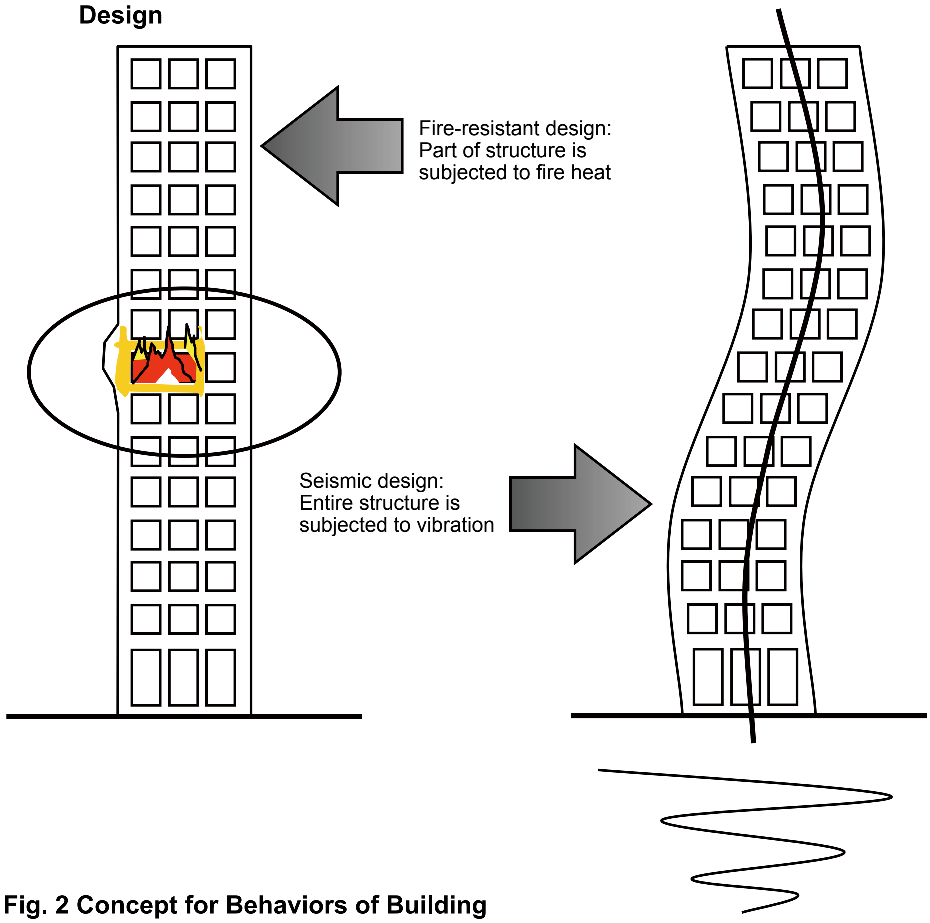
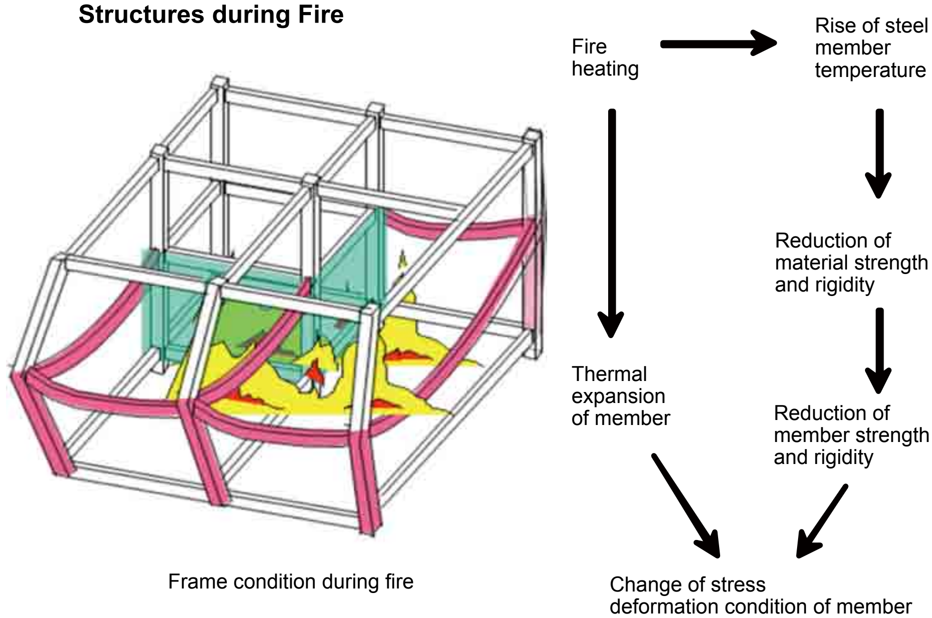


Fig. 2 Concept for Behaviors of Building Structures during Fire



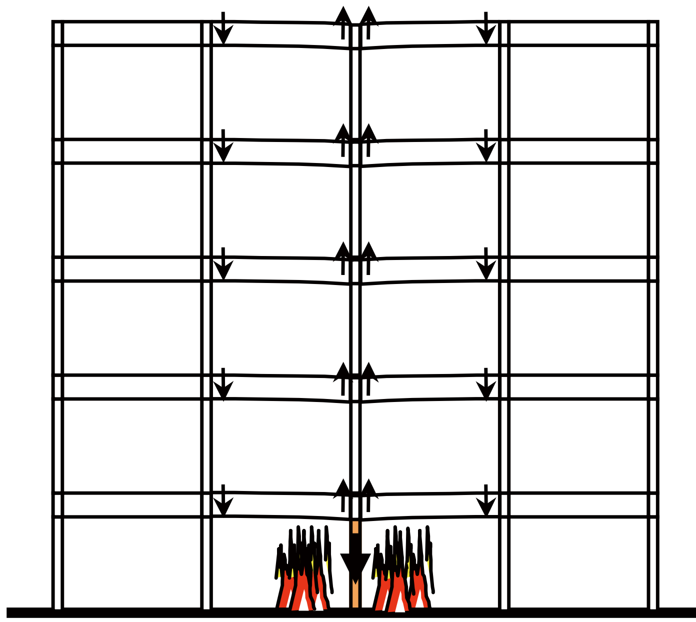
resistance of the columns restricts the thermal expansion, thereby providing bending rigidity that easily causes flexural deformation in the columns. Accordingly, an important factor in examining frame stability during a fire is the total length of the beams used in one layer. Then, because the beams progressively lose rigidity as the temperature rises, they start to undergo deflection due to the vertical load. At the same time, the columns suffer strength reduction due to the effect of the heat from the fire.

At this stage, in common framing structures, even when a group of members located in the section where a fire breaks out lose their strength, the surrounding members that stand outside the area of the fire retain their strength and thus are able to redistribute the stress and prevent a total framing collapse. Fig. 3 shows an image of the stress redistribution. In the framing shown in the figure, even if a fire breaks out on the lower floor and causes some of the columns to lose strength, the axial forces previously borne by these columns is borne by the framing of the upper floor through stress redistribution, thereby preventing the frame from collapsing. In cases when the columns and beams are rigidly joined, stress is easily redistributed. Further, in cases when multiple numbers of beams are installed on the upper floor and there exists an allowance in the sum of the bending strength of these beams, the possibility of frame collapse is further reduced.

Because seismic resistance is required for approval in building construction in Japan, rigid joining is commonly adopted for beam-column connections. Further, in order to treat the horizontal force caused by an earthquake, columns and beams are provided with strength exceeding that needed to support the vertical load. Generally there will be some fires after an earthquake, but during an earthquake violent fires that threaten the structural stability of buildings do not occur. On the other hand, when violent fires break out that threaten the structural stability of buildings, it is extremely unlikely that an earthquake will occur during a fire that is capable of threatening the structural stability of a building. Accordingly, the increased strength provided to handle seismic forces serves to enhance the safety factors designed to prevent framing collapse during a fire. It is generally accepted that buildings constructed with a specified seismic resistance have higher fire resistance, and therefore have high framing redundancy vis-à-vis fires.

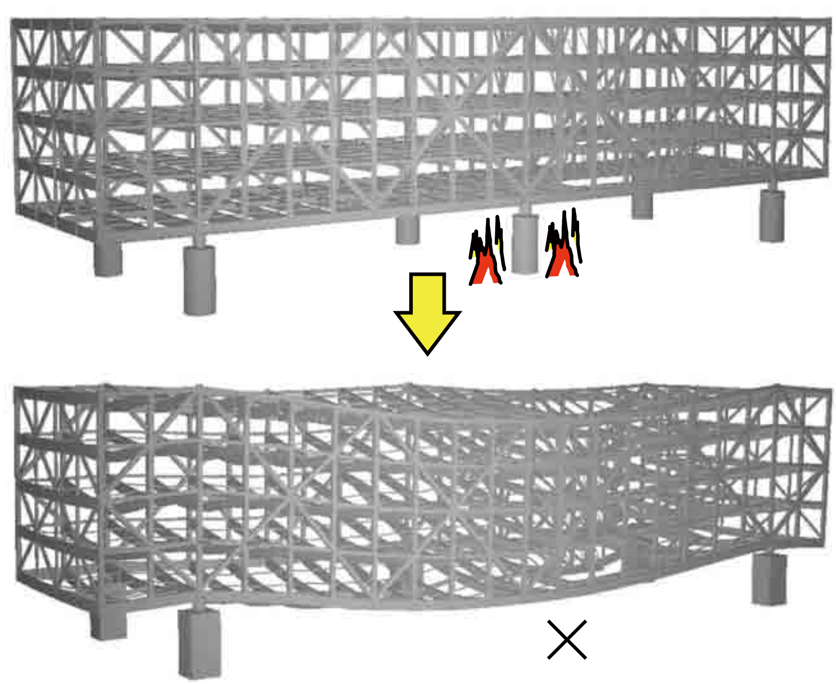
Performance-based fire-resistant design

Fig. 3 Image of Stress Redistribution in Framing



Even when the axial-direction force of the column at the lower floor is lost due to fire, the beam just above the lower floor transfers the force to the right- and left-side frames due to beam's shear strength.

Fig. 4 Condition of Framing when Part of Structural Members Lose Their Strength due to Fire (Indication by increasing deformation magnification)



is practically applied by actively taking into account the redistribution of stress in framing during fires, as mentioned above. In such cases, it is necessary to examine fire resistance by taking into account not only the final fire conditions but fire development as well. In recent building construction, planar and vertical frame composition has become more complex, and in some cases the framing type proper has become complex. In such cases, it is necessary to prepare a framing plan that considers the stress deformation conditions during a fire from the perspective of secular change as well.

Example of Fire-resistant Design Capitalizing on Redundancy

Fig. 4 shows an example of fire-resistant design that takes full advantage of framing redundancy during a fire. The building is composed of mega-framing that adopts diagonal columns at the outer periphery of the building. Loss of strength in the horizontal structural members, such as beams, will lead to collapse and falling of floors; therefore, it is necessary to provide fire protection to these members so that structural strength during a fire can be retained. Even if vertical members such as columns lose strength, the stress borne by these columns can be redistributed to the diagonal columns that compose the mega-framing.

This design example allows the elimination of fire protection for some columns by examining and confirming stress redistribution. The examination, in addition to studying fire development, confirmed two stress conditions: stress in the peripheral frame caused by thermal expansion in the columns; and stress in the peripheral frame caused by loss of strength in the columns. Fire-resistant design, as applied in the case of mega-framing, is becoming frequent because it is easy for the corresponding columns to redistribute stress to the diagonal columns having high strength.

Toward Enhanced Redundancy during Building Fires

For common buildings, "member-level" fire-resistant design is adopted whereby the fire resistance of the entire frame during a fire is secured through securement of the fire resistance of the building's structural members. The use of such fire-resistant design guarantees the fire resistance of the framing of common buildings during a fire. However, while strength at the member level during a fire may have been confirmed, there are cases in which a building suffers total collapse triggered by thermal expansion in the structural members of neighboring buildings during a fire—like the collapse of building WTC7 at the World Trade Center. In the structural design of steel-frame buildings, it will be necessary to work out a structural plan that takes into account structural redundancy during a fire. ■

Full view of Shimizu Institute of Technology of Shimizu Corporation, where R&D is being promoted toward further improvement of fire resistance of buildings (photo courtesy: Shimizu Corporation)

Surveys of Fire Damage of Steel Structures

by Kenichi Ikeda
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Concept for Fire Damage Surveys

Fire causes localized damage in a building due to the heat of the fire. Because the damage occurs in only part of a building, buildings subjected to fire damage are frequently reused. In order to reuse such buildings, it is necessary to understand the level of fire damage and, further, to confirm whether or not the damaged building can tolerate reuse.

A fire damage investigator diagnoses the level of fire damage and proposes from an engineering aspect what kind of repairs and means of reinforcement are required to facilitate the reuse of a damaged building. The target performance to which a fire-damaged building should be restored is decided by the building owner and/or building administrator, and its users.

If the target building is to be restored to previous levels of use, the agreed upon performance at the contract stage must stipulate complete restoration. On the other hand, in cases when the service life of the building is in decline and the building is scheduled for reconstruction in the near future, it is feasible for the structure's target performance to be settled at a level sufficient to remain in service for only a few months. In most cases, the settled target performance is functionally equal to pre-fire levels. With its main focus on targeted performance, the Architectural Institute of Japan has prepared "Guidelines for the Diagnosis of Building Fire Damage and Repair and Reinforcement Methods and Commentary" (draft).

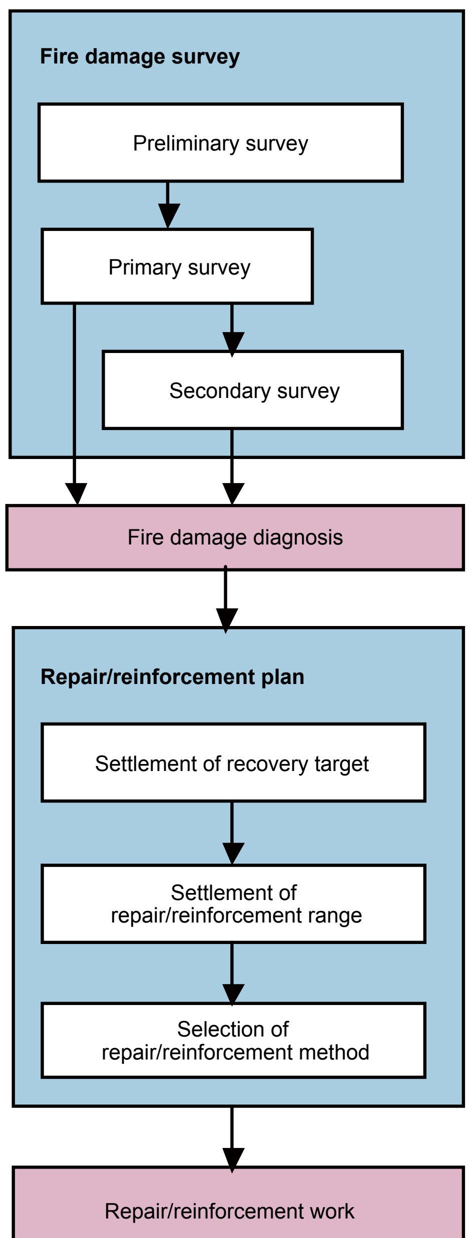
Flow of Fire Damage Surveys

Fig. 1 shows the workflow from the fire damage survey to the repair/reinforcement work. A fire damage survey is composed of the following three parts:

- Preliminary survey: Collection of information about the damaged building from drawings and information about the fire from newspapers and other media prior to visiting the fire site
- Primary survey: Visual on-site survey of the fire conditions
- Secondary survey: Implementation of tests on the structural members extracted from the damaged building, if necessary

Based on the survey results, the range and level of damage due to fire are diagnosed. Then, plans for repairs and reinforcement are worked out based on the fire damage diagnosis. At this stage, settlement is reached regarding the target performance to be recovered, the method of recovering the performance is selected, and the repair and reinforcement works are implemented.

Fig. 1 Flow from Fire Damage Survey to Repair/Reinforcement Work



Figs. 2 and 3 show the workflow in judging the level of fire damage in steel-structure buildings. In the case of surveying steel-structure buildings, not only the lowering of material strength due to fire heat but also the deformation of members and frames due to thermal expansion greatly affect judgments regarding reuse. Accordingly, estimating the heated temperature of the steel-frame members and measuring the deformation of these members compose the main tasks in fire damage surveys.

Estimating the heated temperature of the steel structural members is made in order to assess the changes in the material characteristics of the steel frame members targeted for reuse. Changes in the mechanical properties of high-strength steel products and high-strength bolts, whose strength is enhanced during the manufacturing process, occur at comparatively low temperatures. Even among steel structural members that are barely affected by the heat of a fire, some members suffer large deformation due to the thermal expansion of other steel members. For steel-structure buildings, the heated temperature and member deformation are surveyed to diagnose the fire damage. ■

Fig. 2 Workflow of Steel-frame Repair and Reinforcement Based on Estimated Heated Temperature

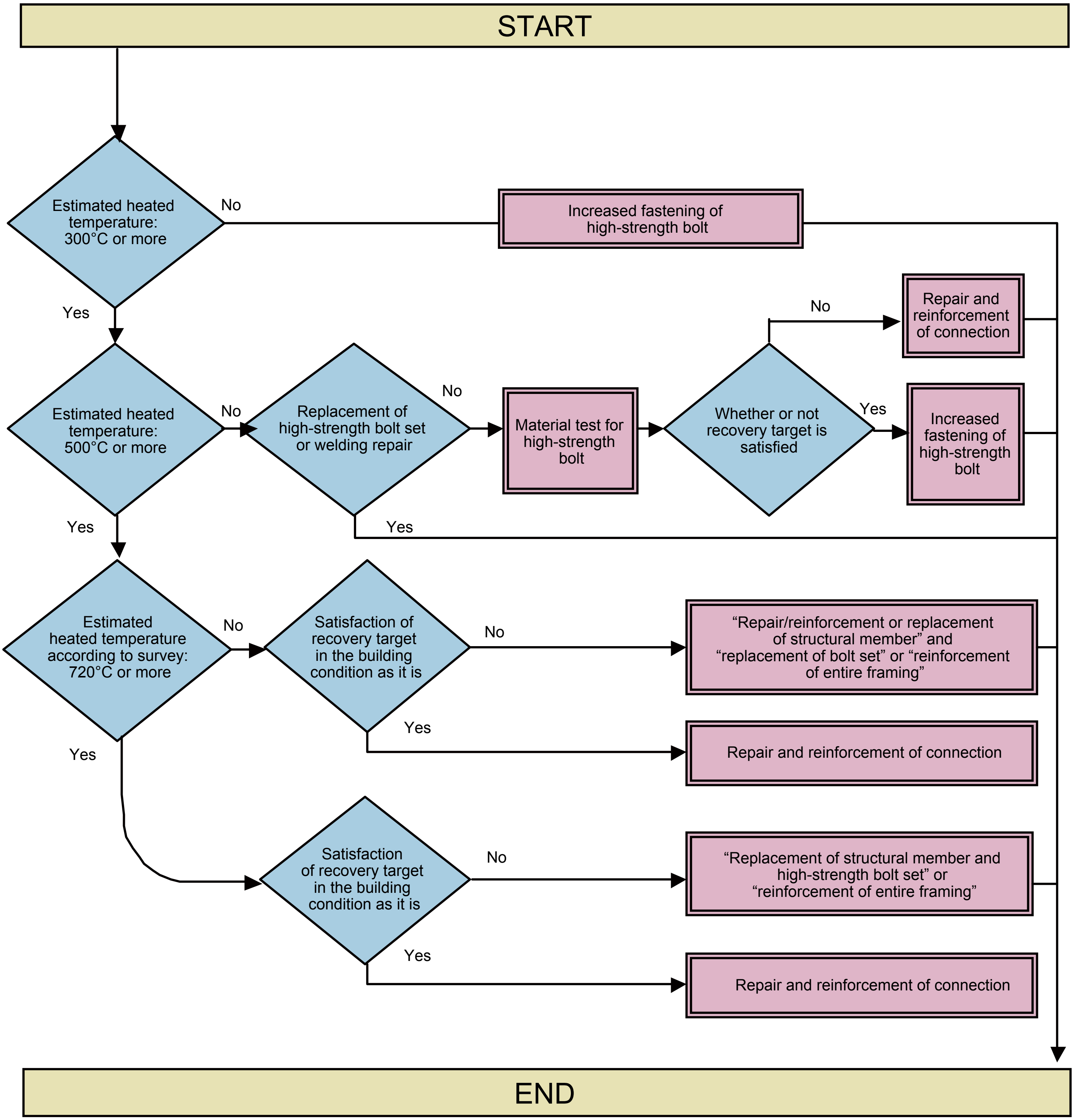
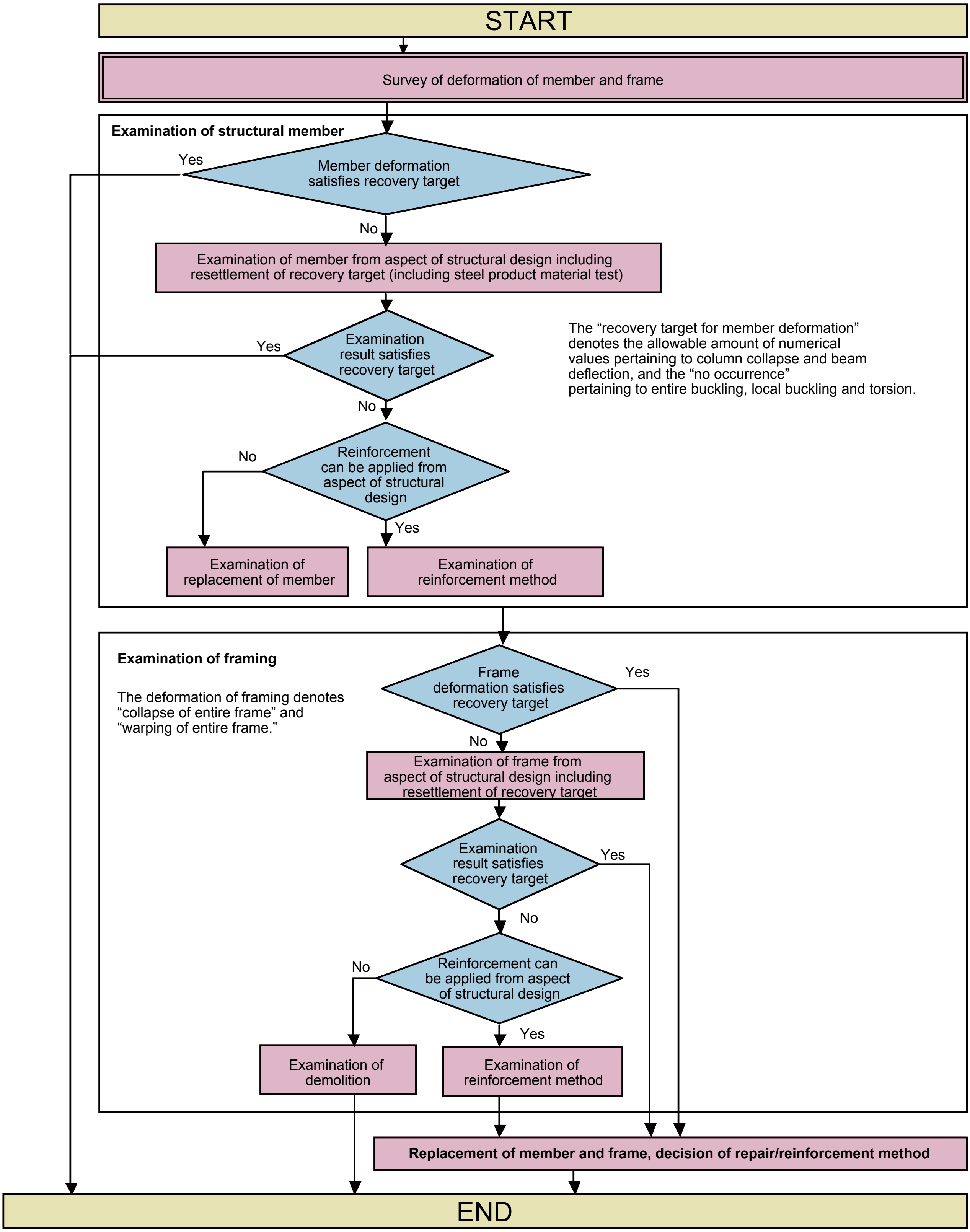


Fig. 3 Workflow of Steel-frame Repair and Reinforcement Based on Deformation



Reference

"Guidelines for the Diagnosis of Building Fire Damage and Repair and Reinforcement Methods and Commentary" (draft), Architectural Institute of Japan, February 25, 2010

Fire-resistant Design of TOKYO SKYTREE

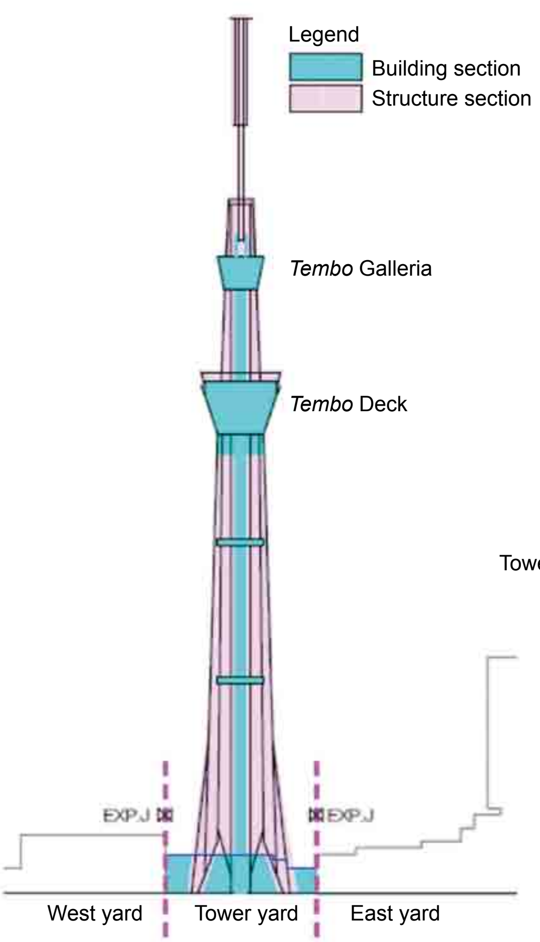
by Tomoyuki Someya, Structural Design Dept., Nikken Sekkei Ltd.

The TOKYO SKYTREE® is a freestanding broadcasting tower that opened in May 2012 in Tokyo. It has a maximum height of 634 m, and serves primarily as both a broadcasting tower and an observatory tower. According to the provisions of the Building Standard Law, the tower is classified as a building in a structure (Fig. 1). The building comprises a low-rise section with commercial zones and a tower section centered on the observatory.

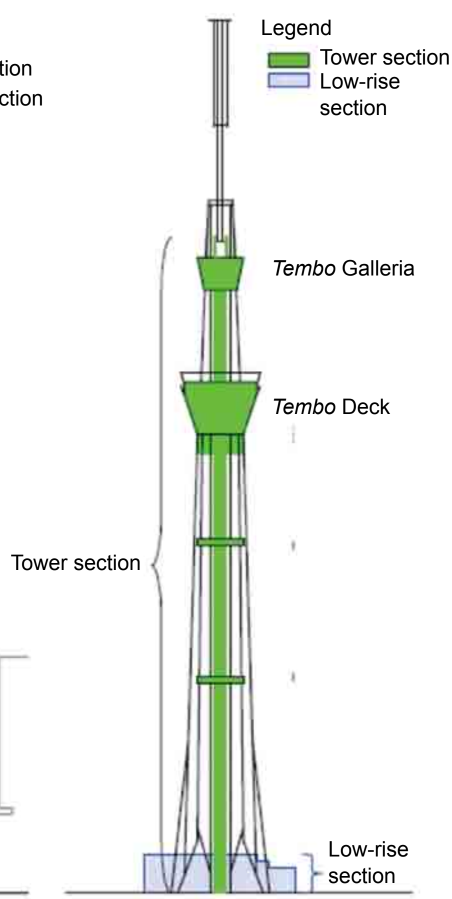
When a fire breaks out in a building, the first priority is to see that all visitors are safely evacuated. A subsequent concern is that a flashover might easily lead to total structural collapse, depending on the scale and location of the building. It is the role of fire-resistant design to prevent this type of building collapse.

In fire-resistant design, it is important to factor in the possibility of such flashover events. Fires that affect a building are classified according to whether they occur inside or outside the building. In the design of TOKYO SKYTREE, both internal and external fire events were assumed, and appropriate fire protection and other means were provided to prevent structural instability in the tower if a fire were to occur.

Fig. 1 Classification as a Building in a Structure, and Tower and Low-rise Sections of TOKYO SKYTREE



Classification as a building in a structure



Classification as tower section and low-rise section

Fire-resistant Design peculiar to the TOKYO SKYTREE

The fire-resistant design adopted for the tower is sufficiently safe against all assumed potential fires that require legal considerations. In addition, because of the tower's importance and its gigantic size, the fire assumptions included in the fire-resistant design surpass the legally required levels, making the building fully prepare for the worst.

One of the fire assumptions that surpass conventional levels pertains to urban fires. An urban fire is assumed to be a conflagration that engulfs the area surrounding a building. As introduced below, the fire-resistant design adopted for TOKYO SKYTREE will not allow structural problems even in such a critical state.

A unique structural feature peculiar to TOKYO SKYTREE is the "glass floor" installed on the *Tembo* (observation) Deck, through which visitors can look directly below them. If the glass floor were to fall out due to a fire, it could cause serious damage in the surrounding area. In order to prevent such an accident, a fire resistance test was conducted by heating a full-scale glass floor model in a furnace to probe whether or not it could withstand the prescribed fire. The test confirmed that no problem would arise, as introduced below.

Proving the Fire Resistance of the Tower Section during an Urban Fire

The first thing done when considering an urban fire is to estimate the incombustibility of the buildings in the target area and then, based on the obtained incombustibility rate, estimate the intensity of fires that might occur in the neighborhood of the building site. Wind velocity and other factors are used to calculate the heated temperature of the section of the tower, and the framing stability at the calculated heated temperature is proven.

Fig. 2 Model to Calculate Heated Temperature due to Assumed Fire and Analytical Example (Wind velocity: 0.5 m/sec)

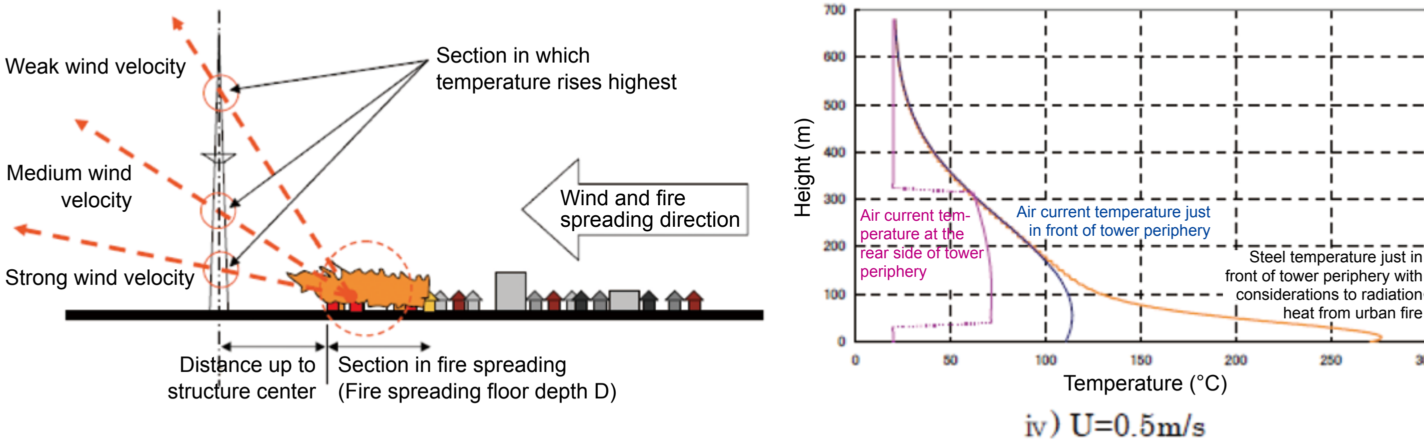


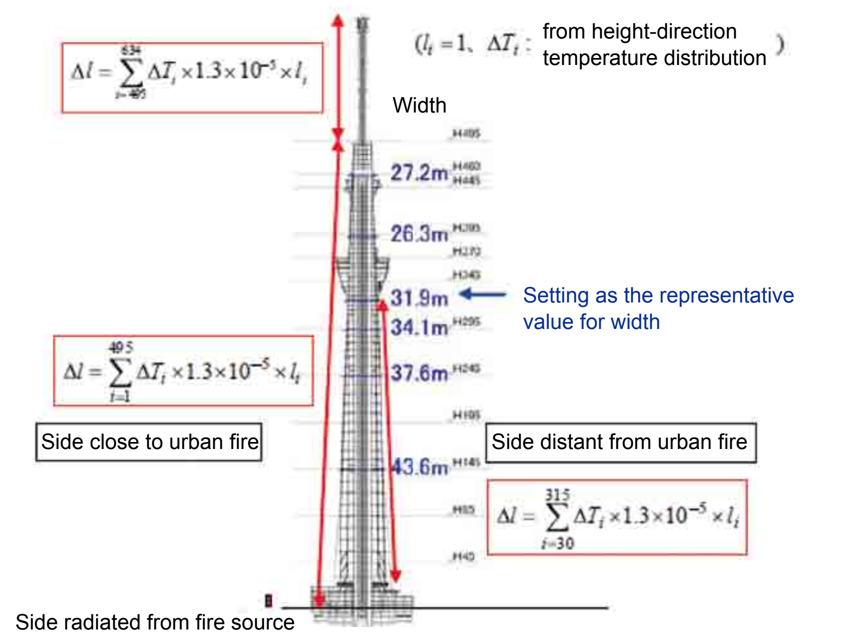
Fig. 2 shows the model used to calculate the heated temperature caused by the assumed fire and the corresponding analytical example. Fig. 3 shows the approach to prove the structural stability of the tower framing against the assumed fire. Even when assuming an urban fire, the respective structural members remain under their elastic limit, and the structural stability of the tower is retained.

Outline of the Fire Resistance Test for the Glass Floor

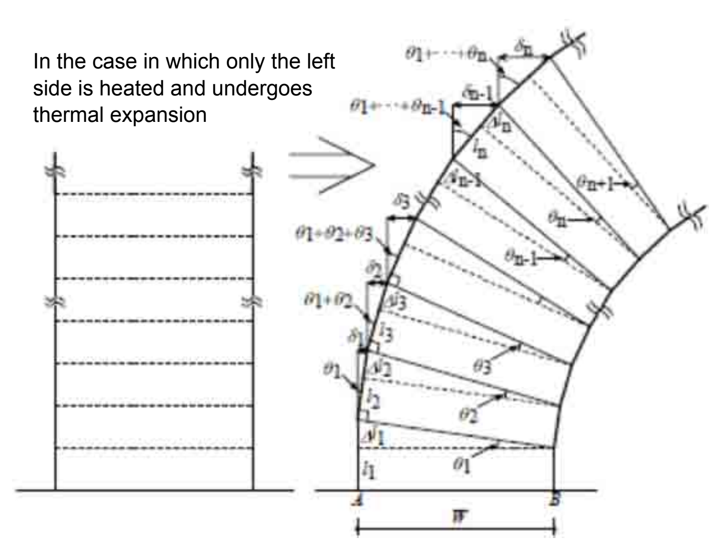
Fig. 4 shows the location and a cross sectional view of the glass floor. The lower section of the floor is for exterior use, and the upper section for interior use. In an internal fire emergency, the upper section would suffer direct exposure to the fire. The fire resistance test was conducted by inserting a full-scale model of the glass floor for interior use into a heating furnace. Photo 1 shows the test specimen. A loaded heating test was adopted in which the specimen was heated with a weight placed upon it. Meanwhile, the five white cylindrical members shown in Photo 1 are the fire-resistive covering of the deformation measurement jig.

Photo 2 shows the specimen under heating. While the assumed duration of the fire inside the building is 36 minutes, a standard one-hour period of heating was applied in the test. Photos 3 and 4 show the specimen after heating. Even in a loaded heating test with a fire duration longer than that of an assumed fire inside the building, cracking occurred in only the two upper layers of the 4-layer laminated test piece, confirming that the glass floor would show sufficient safety during a fire. ■

Fig. 3 Proving of Structural Stability of Tower Framing against Assumed Fire (Concept of Thermal Expansion and Image to Calculate Inter-story Drift)



Concept for thermal expansion range in forecasting tower framing deformation



Method to calculate horizontal displacement

Fig. 4 Location of Glass Floor and Assumed Test Specimen

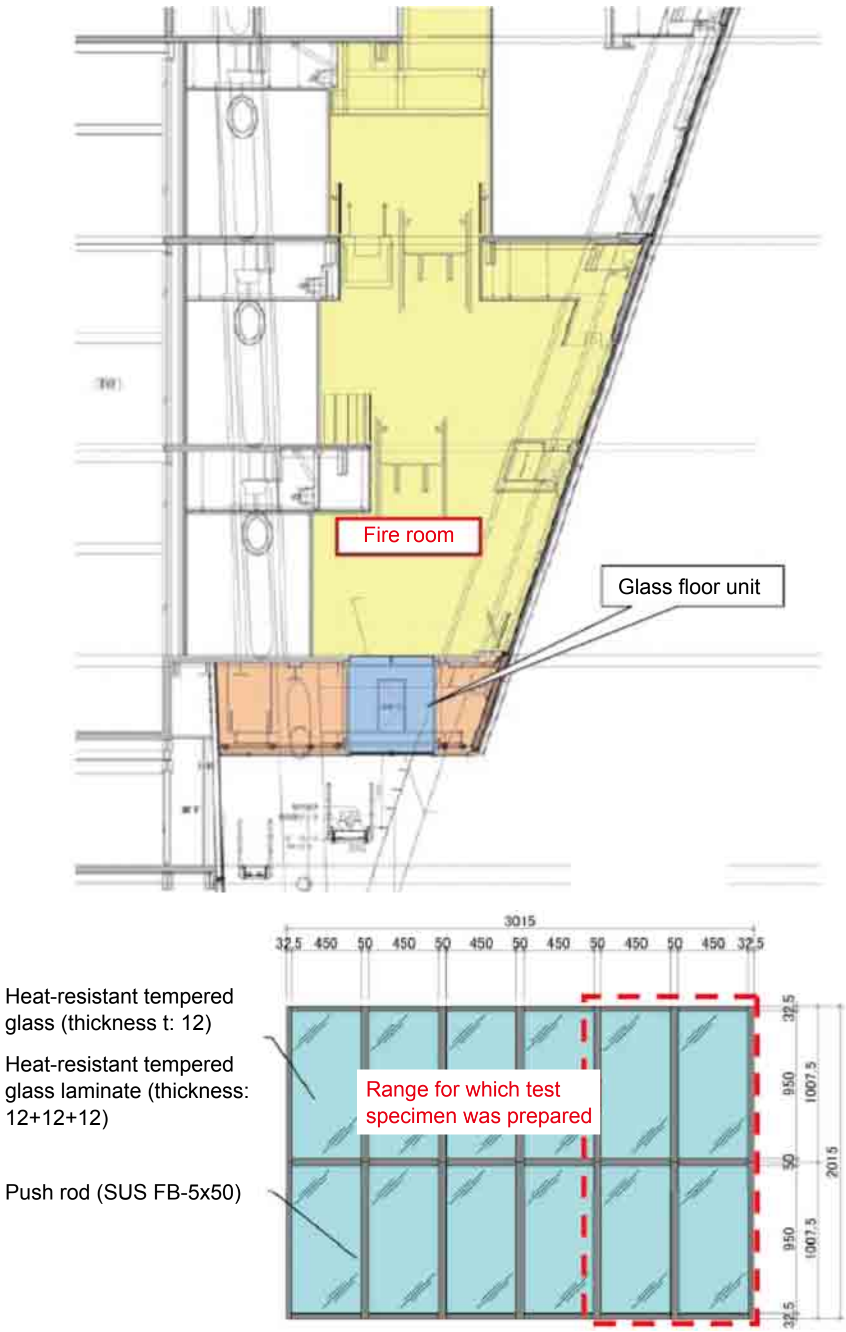


Photo 2 Test specimen in heating (from observation window of test furnace)

Photo 3 Test specimen after heating

Photo 1 Test specimen

Photo 4 After heating (two upper layers were removed to confirm no cracking of third layer of glass laminates)

Fire-resistant Steel

by Committee on Fire Prevention and Fire Resistance
The Japan Iron and Steel Federation

The Building Standard Law of Japan prescribes that special buildings (apartment buildings, hotels and the like) that are used mainly by many and unspecified persons and buildings that are located in urban areas should be of fire-resistant construction. In a fire-resistant structure, it is obligatory that the columns, beams and other major structural sections meet certain fire-resistant specifications or that the fire resistance of these structural members be confirmable by means of calculation.

One of the means to satisfy certain fire resistance specifications in the first option above is to provide fire protection so that the temperature of steel products during a fire does not rise. Because the temperature of a fire is generally around 1,000°C, it has conventionally been necessary to add fire protection to general steel that is heat-insulated to withstand temperatures up to 350°C. However, with fire-resistant (FR) steel, the heat insulation is adequate up to 600°C, which allows for a significant reduction in the use of fire protection. In addition, in cases where the fire conditions and building design conditions during a fire keep the temperature of the steel products below 600°C, FR steel can be applied without fire protection. That is, the application of FR steel lowers construction costs, reduces construction time and allows for more effective use of indoor space.

Excellent High-temperature Properties of FR Steel

The high-temperature resistance of FR steel has been greatly enhanced by the addition of appropriate amounts of alloying elements such as Mo, Nb and Cr and by controlling the heat-treatment conditions. The high-temperature yield strength of FR steel has been improved by means of the precipitation and strengthened distribution of carbon nitrides, and by strengthening the solution treatment of the alloying elements. Among the features of FR steel are:

- High-temperature strength is excellent, and the yield strength at 600°C (0.2% offset) is more than 2/3 that of the specified room temperature value.
- The room temperature properties conform to the specifications for numerous kinds of structural steel products.
- Weldability is similar or superior to that of general steel.

Among the related standards that specify the high-temperature properties of steel products is ASTM A 1077 (Standard

Example of application of FR steel in open-type multi-storied parking garage

Fig. 1 Temperature Dependency of High-temperature Yield Strength

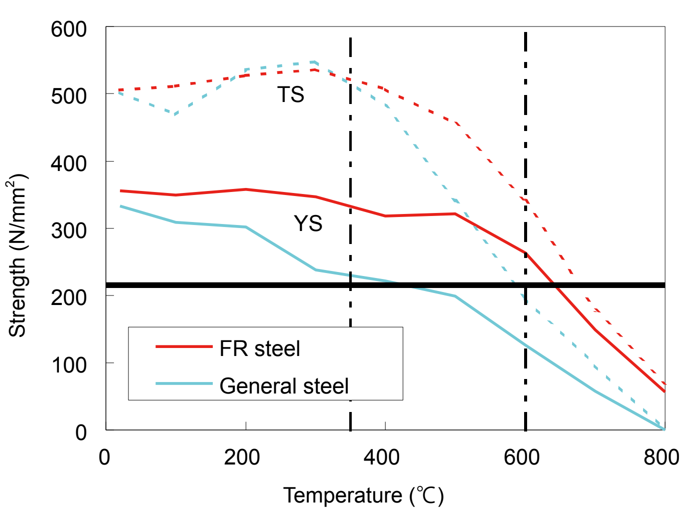
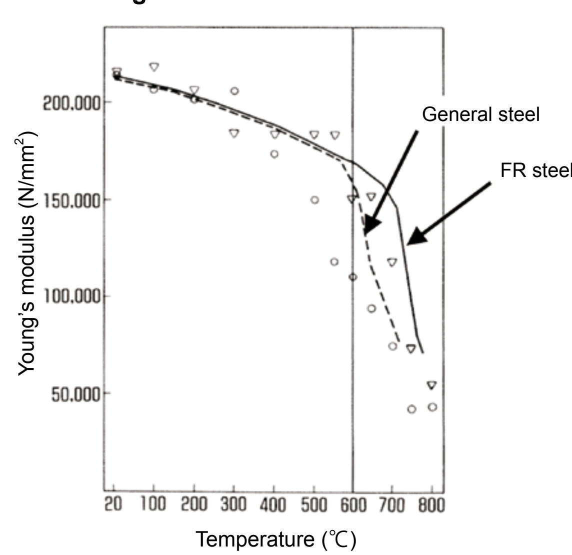


Fig. 2 Temperature Dependency of High-temperature Young's Modulus



Specification for Structural Steel with Improved Yield Strength at High Temperature for Use in Buildings) that was issued in April 2012. This standard requires advance confirmation from the respective steelmakers as to whether or not they can produce steel in conformance with the standard.

Material Properties of FR Steel

Fig. 1 shows a comparison of the temperature dependency of high-temperature yield strength between FR steel and general steel. While the yield strength of general steel declines around 350°C to 2/3 the specified value at room temperature, that of FR steel remains at 2/3 or more until the temperature surpasses 600°C, which demonstrates the superior high-temperature properties of FR steel compared to general steel. Fig. 2 shows a comparison of the temperature dependency of the high-temperature Young's modulus between FR steel and general steel. The decline of the modulus at 550°C~700°C for FR steel is smaller than for general steel.

Member Properties of FR Steel

A loaded heating test was conducted to confirm that the high-temperature properties of FR steel remain intact when used as columns, beams and other structural members, an outline of which is introduced.

The test method adopted was the loaded heating test for columns prescribed in ISO834. The test was conducted using a large-scale fire resistance test furnace for columns at the Building Research Institute of the Ministry of Land, Infrastructure, Transport and Tourism. The FR steel test specimen was H-300×300×10×15 with a length of 3.5 m (Fig. 3). Blanket-type fire protection was used with the specimen. Meanwhile, another specimen using ordinary steel of identical dimensions was also tested for a comparison of their properties. Fig. 4 shows the relation between the heating time and the steel product temperature. The time up to the collapse of the FR steel column is longer than for the general steel column, and the temperature of the FR steel column at which collapse occurred is higher. It is understood from the figure that even when used as a structural member, FR steel demonstrates better higher-temperature properties than general steel does.

Joining Materials for FR Steel

In the welding of structural members made of FR steel, the welding materials are prepared exclusively in order to secure a high-temperature strength for the welds that is similar or superior to that of the base metal. The characteristic properties of weld joints made using welding materials for FR steel are similar or superior to those of general steel, and the high-temperature tensile strength of the weld joints is also similar or superior to that of the base metal.

Further, in the high-strength bolt joining of major structural sections employing FR steel, the high-strength bolts used for FR steel are designed to secure a high-temperature yield strength for the bolt joints that is similar or superior to that of the structural members. Tor-shear-type high-strength bolts, high-strength hexagonal bolts and galvanized bolts, respectively, are available for use with FR steel, and are used for bolt joining in the same manner as for conventional steel. Fig. 5 shows an example of the tension test results for the base metal used in high-strength bolts. Test results show that high-strength bolts for FR steel are fire resistant at 600°C, about twice as high as high-strength bolts for conventional steel. ■

Fig. 3 Outline of Fire Resistance Test

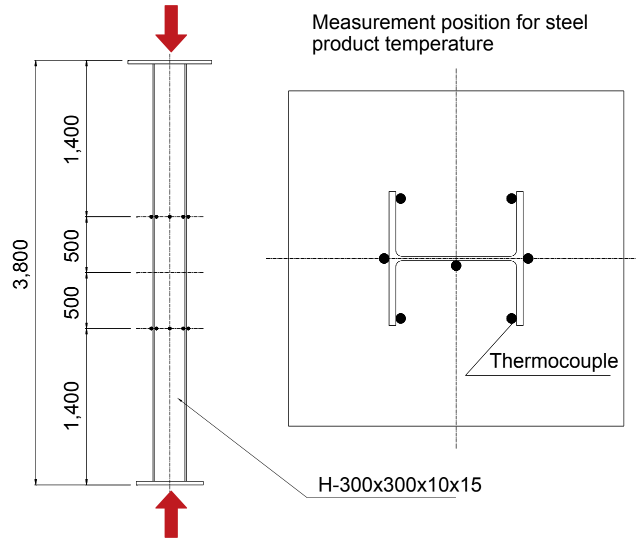


Fig. 4 Relation between Heating Time and Steel Product Temperature

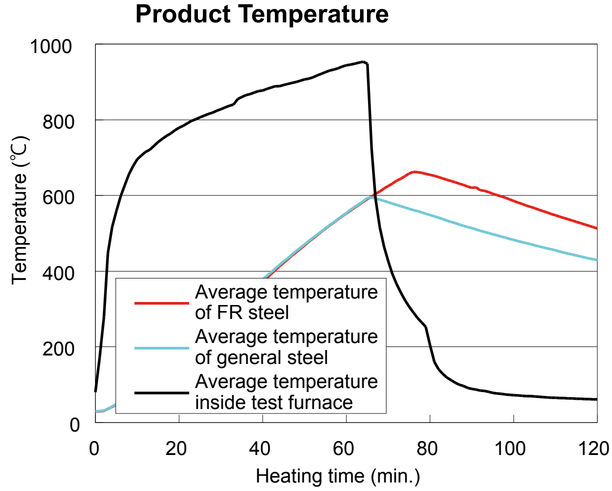
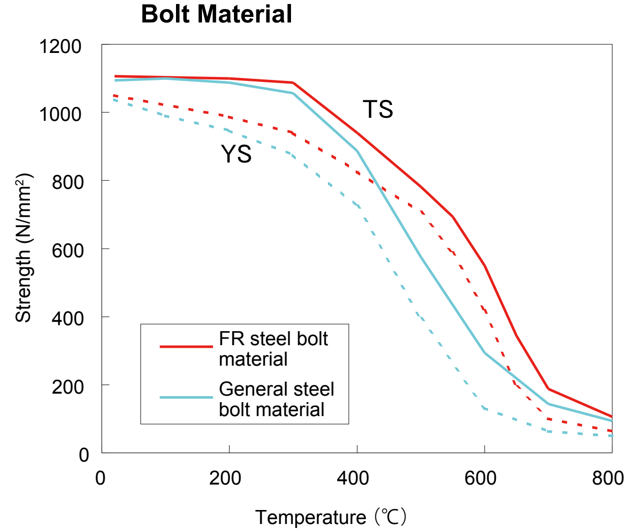


Fig. 5 Results of Tension Test for High-strength Bolt Material



Basic Details about Welding and Welding Control

—On-site Welding—

by Tadao Nakagomi
Professor, Department of Architecture
Shinshu University

For Japan with its frequent great earthquakes, seismic design is very important. Further, there are cases in which welds can become the starting point of fractures that lead to the collapse of steel structures. In the No. 37 issue (December 2012), basic details about welding and welding control in steel-frame building structures and information regarding the key dynamic performance characteristics of welds were introduced. The current issue introduces a discussion about on-site welding, mainly the problems that occur in on-site welding and appropriate countermeasures.

On-site Welding

In the welding of steel-frame structures, two methods—shop welding (non-scallop method) and on-site welding (scallop method)—are used. In Japan, shop welding is more extensively adopted. Figs. 1 and 2 show examples of both shop and on-site welding. In the scallop method, the inward preparation of the grooves allows welding to be done both on the construction site and in the shop; and, the backing metal is attached outside the members. On the other hand, in the non-scallop method, the grooves are prepared outward, which means that welding can be applied only in a flat position and that this method, therefore, cannot be applied at the construction site. In addition, in this method the backing metal is attached inward.

In the construction of giant structures where it is difficult to adopt the non-scallop method, which requires that the shop-welded structural members be transported to the construction site, the scallop method is adopted because it does permit on-site welding. However, in cases when a steel-structure building is subjected to large external forces, such as seismic forces, the strength and deformation capacities of the on-site scallop welding method, which produces sectional defects (scallops) at the web, fall much lower than those of the shop-welding non-scallop method because the stress concentrates at the scallop bottom.

Measures to Improve Deformation Capacity

As mentioned above, in on-site welding, strength and deformation capacity decrease. Therefore, it is necessary to take appropriate measures to improve the deformation capacity, three methods of which are introduced below. Tests were conducted to confirm the effectiveness of these measures. Fig. 3 shows the shape and installation position of the test specimens, Table 1 shows the test results, and Figs. 4, 5 and 6 show the relation between the load and the displacement of the on-site scallops.

Fig. 1 Shop Welding Non-scallop

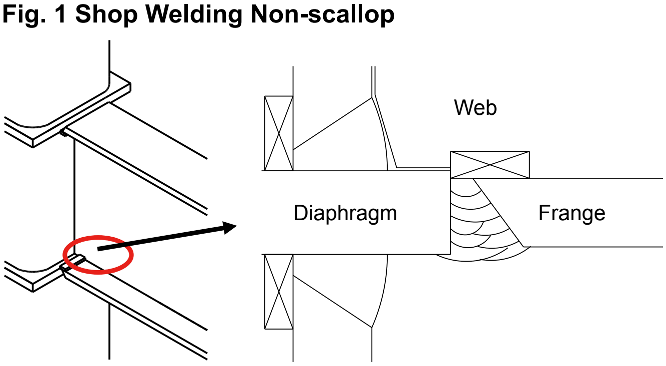


Fig. 2 On-site Welding

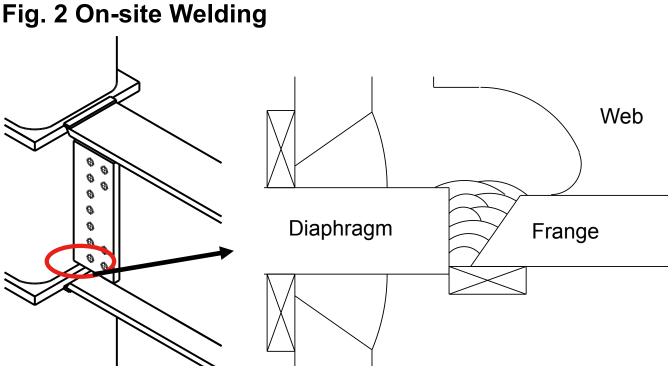


Fig. 3 Test Specimen Configuration and Installation

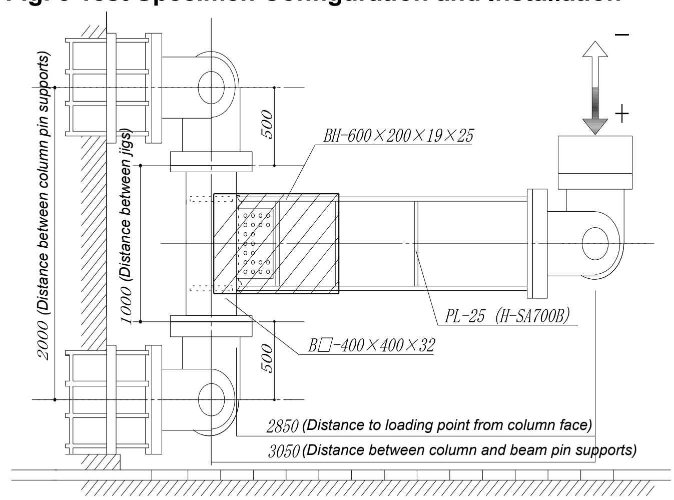


Fig. 4 Load-Displacement Relation of On-site Welding Scallop

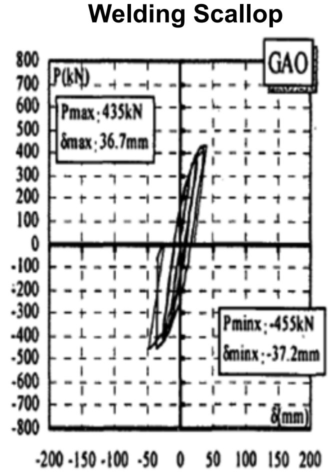


Fig. 5 Load-Displacement Relation of On-site Welding Scallop

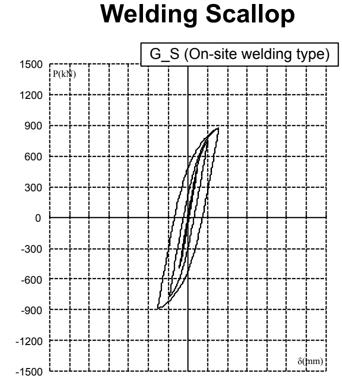
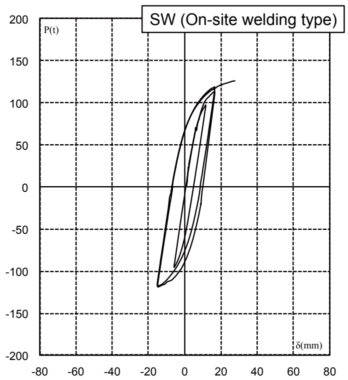


Fig. 6 Load-Displacement Relation of On-site Welding Scallop



• Drilled Flange Method

This method aims at distributing the stress in column-beam welds to holes drilled in the flanges so as to reduce the stress at the beam end. Fig. 7 shows the configuration of the drilled flange and Fig. 8 shows the relation between the load and the displacement.

Fig. 7 Drilled Flange

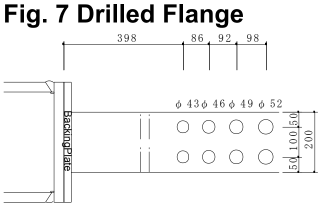
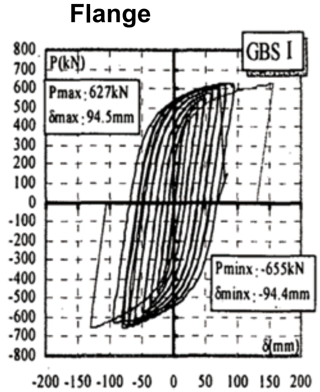


Fig. 8 Load-Displacement Relation of Drilled Flange



• Flange Width Enlargement Employing a Haunch

This method aims not only at reducing the stress occurring in the beam end by means of widening the flange sectional area attained by enlarging the beam connection width but also at transferring the position of maximum stress to the beam base metal. Fig. 9 shows the haunch, and Fig. 10 the relation between the load and the displacement.

Fig. 9 Haunch

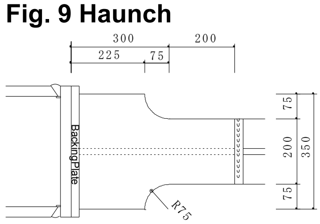
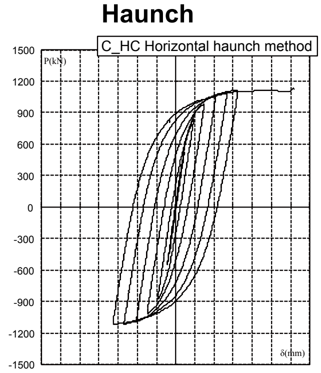


Fig. 10 Load-Displacement Relation of Haunch



• Method to Provide Reinforcement Bead to Scallop Bottom

In order to improve deformation capacity, this method aims at relaxing the stress concentration by providing a reinforcement bead to the scallop bottom, which would be the starting point of a fracture in the non-scallop method. Fig. 11 shows the reinforcement bead, and Fig. 12 the relation between the load and the displacement. Meanwhile, the size of the test specimen was similar to that used in the two methods mentioned above, but the test method differed from the above two, and a 3-point bending test was adopted.

Fig. 11 Reinforcement Bead

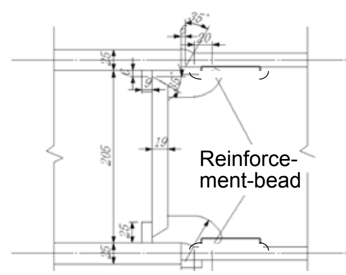
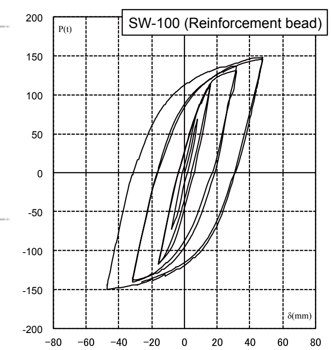


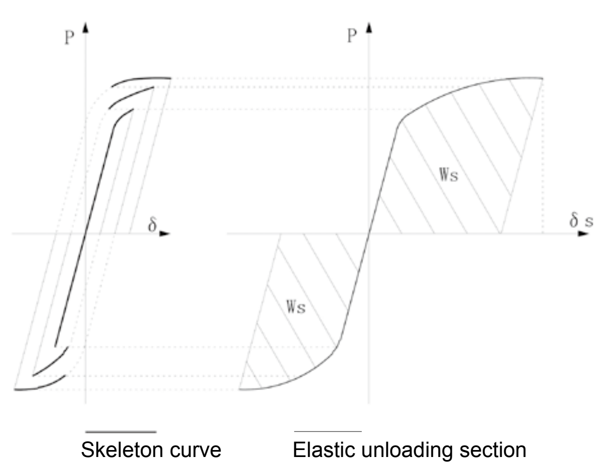
Fig. 12 Load-Displacement Relation of Reinforcement Bead



Method to Calculate *η*s

Fig. 13 shows the general P-δ relation of a steel structural member subjected to cyclic bending. The skeleton curve corresponds to a loading sphere that surpasses the maximum strength so far demonstrated by the steel member. Because it is shown in existing research that the area produced by joining together the skeleton curves is equivalent to the P-δ curve of a member subjected to monotonous loading, the skeleton curve can serve as an appropriate parameter in assessing the deformation capacity of a member subjected to random external forces such as seismic loads. The accumulated plastic deformation magnification *η*s is used as the parameter for the deformation capacity and is obtained by dividing the value found by doubling the elastic limit distortion energy (Ws) of the absorbed energy in the skeleton curve by (${}_cP_p \times {}_c\delta_p$).

Fig. 13 Calculation of Skeleton Curve



(Continued overleaf)

Improved Deformation Capacity in On-site Welding

As can be seen in Table 1, the test results show that, while the deformation capacity of conventional on-site welding is low: 1~4, the deformation capacity of an on-site welding method provided with the measures mentioned above is sufficient: 7~12. Further, it can be understood that while the deformation capacity of the shop-welding non-scallop method is 6.9, the on-site scallop method can offer a deformation capacity similar or superior to the shop-welding non-scallop method by providing the measures introduced above. ■

Table 1 Test Results

Test specimen	Loading direction	${}_eP_{max}$ (kN)	${}_e\delta_{max}$ (mm)	η_s
GAO	+	435	36.73	1.30
	–	-455	-37.20	1.60
KAO	+	550	87.96	6.90
	–	-573	-80.49	7.90
GBS	+	627	94.45	12.40
	–	-655	-94.37	9.40
G_S	+	-878	-78.53	2.24
	–	889	75.98	2.91
G_HC	+	-1114	-244.58	11.16
	–	1116	162.95	8.46
SW	+	118.5	16.0	3.14
	–	-118.3	-16.0	2.04
SW-100	+	147.8	47.8	8.94
	–	-149.9	-47.4	9.56

JISF Operations

Recent Operations of the Committee on Overseas Market Promotion

The Committee on Overseas Market Promotion, The Japan Iron and Steel Federation (JISF), promotes diverse operations pertaining to the introduction of high-performance steel products and their application technologies as practiced in Japan. Its main aim is to increase the use of steel structures overseas. Recent operations are introduced below.

Local Surveys in Cambodia

The committee has visited Cambodia three times to make local surveys regarding economic conditions, infrastructure construction, tasks remaining to encourage wider applications of steel structures and tasks that JISF still needs to carry out in that country. In 2012, the "Conference on Advanced Technologies for Steel Construction 2012" was held in Phnom Penh by JISF jointly with the Ministry of Public Works and Transport of Cambodia and the Institute of Technology of Cambodia and with support of the Embassy of Japan in Cambodia. About 200 engineers from governmental, academic and private sectors participated in the conference, producing successful results. The conference was highly assessed by the participants, and according to a questionnaire survey, more than 90% of the participants hoped to participate in the next conference.

Local Surveys in Myanmar

Subsequent to the visit to Yangon made in 2012, the committee members visited a total of six related government agencies not only in Yangon but in Naypydaw, too, for exchanges of ideas and information. These agencies have high expectations for the steel-structure diffusion activities to be promoted by JISF. The committee plans to continue conducting surveys on steel-structure diffusion in the future. ■

"Conference on Advanced Technologies for Steel Construction 2012" held for the first time in Phnom Penh in December 2012

H.E. TAUCH Chankosal, Secretary of State of MPWT (center) and Mr. Yoshihiro Higuchi, Minister Deputy Chief of Mission, Embassy of Japan in Cambodia (back right)

Lecture delivery

Closing remarks by Dr. OM Romny, Director General, Institute of Technology of Cambodia

Exchange meeting in Myanmar: (from left to right) Meetings at the Ministry of Construction, the Ministry of Rail Transportation and the Ministry of Transport

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