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Feature Articles

Advanced Design of Structural Steel Buildings in Japan

Serial Article: Latest Design of Steel Buildings in Japan (2)

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Publication of English Translation of the 2005 AIJ Design Standards for Structural Steel Buildings

Organization of Design Standards and Recommendations for Structural Steel Buildings in Japan

by Motohide Tada
Professor, Osaka University

Motohide Tada: After finishing the master's course of the Graduate School of Engineering, Osaka University, he entered Nikken Sekkei Ltd. in 1982. He became assistant professor of the School of Engineering, Osaka University in 1989 and assumed his current position as professor of the School of Engineering, Osaka University in 2007. His specialization is building structures.

Three Design Methods for Structural Steel Buildings

Fig. 1 illustrates the organization of design standards and recommendations for structural steel buildings prescribed by the Architectural Institute of Japan (AIJ). The structural design methods adopted in Japan is classified into three distinct methods—allowable stress design, plastic design and limit-state design. For structural steel buildings, one design standard and two recommendations are prescribed by AIJ in compliance with these three methods as follows:

- Allowable stress design addressed by *Design Standard for Steel Structures—Based on Allowable Stress Concept*

For each member comprising the structure, the stresses computed for temporary and sustained load combinations (the two load intensities defined based on frequency of occurrence) must be no greater than the respective allowable stresses. Implicit in this design method is the premise that the building structure remains elastic for temporary and sustained loads.

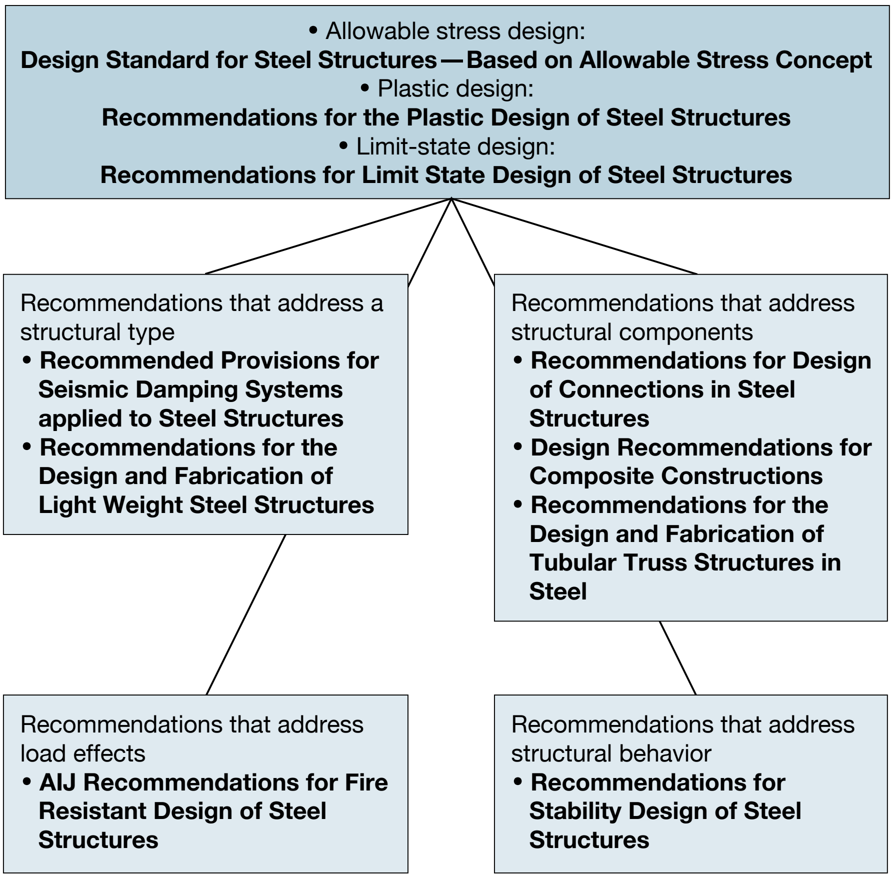
- Plastic design (ultimate load design method) addressed by *Recommendations for the Plastic Design of Steel Structures*

The members are designed so that the load that causes frame collapse (collapse load) surpasses the ultimate load obtained by multiplying the design load by a load factor. This design method permits controlled damage caused by the structural system deforming beyond its elastic limit.

- Limit-state design addressed by *Recommendations for the Limit State Design of Steel Structures*

This method is established based on two key features. First, design requirements are specified for limit states. Limit states are conditions beyond which functionality of the structural system or member cannot be maintained or fundamental assumptions are invalid. Second, design loads and member strengths are determined based on a probabilistic approach. There are two fundamental limit states: the "strength limit state" which addresses structural safety under extreme loading conditions; and "serviceability limit state" which address-

Fig. 1 Organization of AIJ Design Standards and Recommendations for Structural Steel Buildings



es usability, functionality and habitability of the building for daily use.

Supplementary Recommendations to the Three Major Design Systems

AIJ provides a number of design recommendations that supplement the design standard and recommendations mentioned above. The design recommendations may be categorized into the following four types:

- Recommendations that address a specific structural type (*Recommended Provisions for Seismic Damping Systems Applied to Steel Structures,* and *Recommendations for the Design and Fabrication of Light Weight Steel Structures*)
- Recommendations that address specific structural components (*Recommendations for Design of Connections in Steel Structures, Design Recommendations for Composite Constructions,* and *Recommendations for the Design and Fabrication of Tubular Truss Structures in Steel*)
- Recommendations that address specific load effects (*AIJ Recommendations for Fire Resistant Design of Steel Structures*)
- Recommendations that address a specific structural behavior such as buckling (*Recommendations for Stability Design of Steel Structures*)

Characteristic features and roles of each design recommendation are summarized in the following by excerpting and quoting from the preface of each recommendation.

• Recommended Provisions for Seismic Damping Systems Applied to Steel Structures

This document addresses steel structures employing columns and beams as a primary structural system and buckling-restrained braces and/or shear panels as supplemental damping systems. The provisions present performance assessment methods for commonly used steel dampers, analytical modeling procedures for dampers and response-controlled steel structures, and design methods that assure that the story-drift angles of response-controlled steel structures remain within the target limit under design ground motions. (Excerpted from preface of the first edition)

• Recommendations for the Design and Fabrication of Light Weight Steel Structures

This document addresses steel structures with three or fewer stories composed of steel whose plate thickness is no greater than 6 mm. The 1985 revision assures the same structural performance and reliability expected from the capacity design method introduced by the 1981 revision of the Building Standard Law of Japan. Based on the realization that plastic analysis is not directly applicable to light-gage steel structures, the recommendations propose to adjust the first-stage design procedure (achieved by allowable stress design, and whose primary purpose is to assure no damage from smaller and frequent earthquake loads) to replace the second-stage design procedure (achieved by ultimate-strength design, and whose purpose is to assure formation of a controlled energy dissipating mechanism against extreme earthquake loads). (Quoted from preface of the 1985 edition)

• Recommendations for Design of Connections in Steel Structures

Provisions and requirements for welded connections, bolted connections, and column bases, which had been stipulated in separate AIJ standards and recommendations, are assembled in this single document with a comprehensive and extensive commentary. Two strengths are specified for each connection type: the elastic-limit strength and ultimate strength. The elastic-limit strength is adequate for allowable stress design for temporary loads. The ultimate strength represents the maximum force that the connection can transfer. In combination with an adequate design methodology, these connection strengths provide the fundamental design basis for structural steel buildings. (Excerpted from preface of the first edition)

• Design Recommendations for Composite Constructions

This document is composed of four parts, Part 1: Structural Design Recommendations for Composite Beams; Part 2: Structural Design Recommendations for Deck Composite Slabs; Part 3: Design Recommendations for Steel Frame-Reinforced Concrete Load-Bearing Wall Composite Structures; and Part 4: Design Recommendations for Anchor Bolts. The recommendations have responded to immediate needs for rational design methods and seismic upgrade schemes employing composite construction. (Excerpted from preface of the first edition)

• Recommendations for the Design and Fabrication of Steel Tubular Truss Structures

This document addresses design and fabrication requirements that are specific to steel pipes and tubes comprising truss structures. Because members for this committee have been active in the X-VE Technical Sub-commission of the International Institute of Welding, fundamental elements of the recommendations agree with many overseas specifications. (Quoted from preface of the 2002 edition)

• AIJ Recommendations for Fire Resistant Design of Steel Structures

This document reexamined the fire-resistant design provisions prevailing in the Japanese laws and ordinances (as of 1999) that are based on allowable member temperature and required fire endurance duration. The recommendations present a rational, simple, and realistic design framework that is based on the ultimate strength concept which compares structural strength against load effects. (Excerpted from preface of the first edition)

• Recommendations for Stability Design of Steel Structures

This document serves four purposes: first, to clarify the technical basis of buckling-related provisions contained in various specifications on structural steel buildings; second, to explain the concepts related to buckling phenomena and to clarify how the phenomena relate to design; third, to serve user convenience by compiling equations and design methodologies that enjoy popular practical usage; and fourth, to aid starting engineers and students by presenting practical examples. (Excerpted from preface of the first edition)

♦♦♦

The organization, role, and contents of the AIJ standards and recommendations on structural steel buildings was introduced in this article. ■

Now Available: English Translation of the 2005 AIJ *Design Standard for Steel Structures*

by Taichiro Okazaki
Professor, Hokkaido University

Taichiro Okazaki: After finishing the doctor's course at the Graduate School of Engineering, Kyoto University in 1996 and receiving Ph.D. from the University of Texas in 2004, he became assistant professor, University of Minnesota in 2005. Then he served as researcher, National Research Institute for Earth Science and Disaster Resilience in 2009 and assumed his current position as professor, Hokkaido University in 2016. His specialization covers steel structures and earthquake engineering.

An English translation of the 2005 AIJ *Design Standard for Steel Structures—Based on Allowable Stress Concept*, hereinafter referred to as the *Standard*, is available for download from the Digital Contents Distribution webpage (https://www.aij.or.jp/eng/publish/index_ddonly.htm) of the Architectural Institute of Japan (AIJ). The translation (front cover shown in Photo 1) was produced by the Sub Committee to Prepare English Versions of Design Provisions for Steel Structures whose members include researchers in structural steel buildings and representatives of Japanese steel producers. AIJ intends to make this translation the first of a series of English editions of its design specifications for structural steel buildings described in the first half of this article.

The *Standard* prescribes the most fundamental design rules for structural steel buildings constructed in Japan. Since 1981, the building regulations in Japan have comprised a two-level design procedure requiring allowable stress design for moderate earthquake loads and ultimate strength design for severe earthquake loads. As implied by the title, the *Standard* applies to the former design procedure which dictates the proportion of structural members for the vast majority of ordinary steel buildings. While the ultimate strength design is required for high-rise, long-span and other special buildings, the allowable stress design is a general rule that is required for all buildings regardless of height or structural type or configuration.

The translation includes the main body of the *Standard* and Special Commentaries for the English Edition. The Special Commentaries are intended to aid readers who are not familiar with the regulations, codes and provisions, or design and construction practice in Japan. Therefore, the translation may be used not only as a stand-alone design standard but also as a source of information for structural steel buildings in Japan. For example, the relationship between legal design regulations and AIJ technical documents are described in the preface and elsewhere as appropriate. The different types of structural steel, listed in Table 1, and the definition of *F* value, or standard allowable stress, are explained in the Special Commentaries of Section 5.1. The allowable strengths of structural bolts are described in Section 5.2.

The *Standard* was first published in 1970 and updated in the latest edition published in 2005. The Sub Committee recognizes that the *Standard* owes significantly to the *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings* published by the American Institute of Steel Construction (AISC), in particular to the 1963 and 1967 editions. The chapter organization and many provisions of the first edition of the *Standard* were taken from the AISC *Specification*. The primary difference of the AIJ *Standard* from the AISC *Specification* has been the premise that seismic loads are dominant at any geographic location in Japan.

The 2005 edition incorporates up-to-date scientific knowledge and current Japanese practice in member strength (Chapter 5), design for fatigue (Chapter 7), bolts (Chapter 15), welds (Chapter 16), and column bases (Chapter 17). Unique features of the *Standard* that may not be seen in other international design

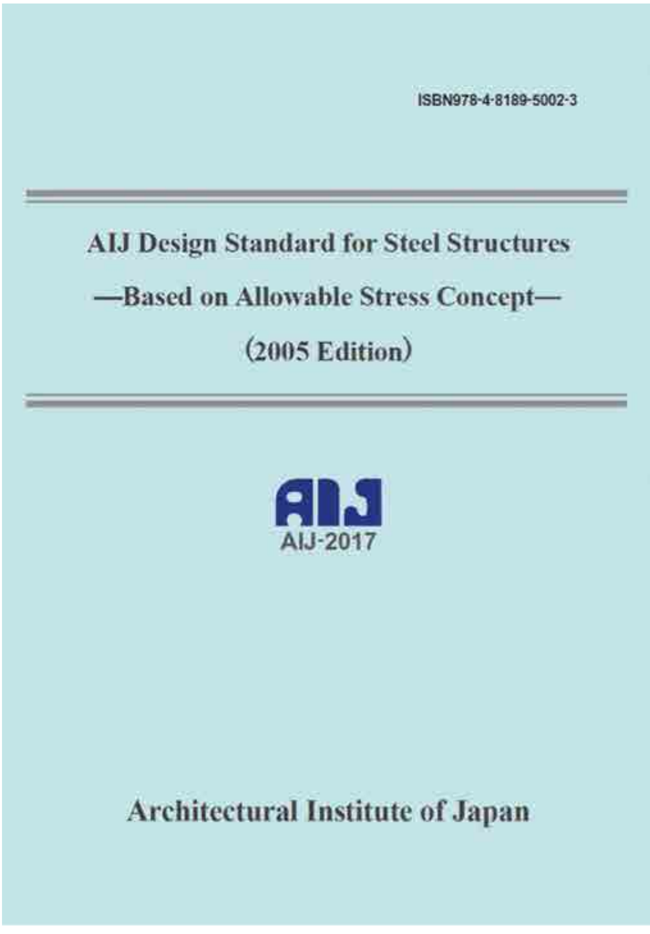


Photo 1
Front cover of 2005 AIJ *Design Standard for Steel Structures*

standards and regulations include comprehensive coverage of built-up sections with web openings (Section 9.2 for beams, Section 11.6 for general compression members, and Section 11.10 for columns) and design requirements for the three general types of column bases (Section 17.2 for exposed type, Section 17.3 for encased type, and Section 17.4 for embedded type).

We are hopeful that the English translation of the *Standard* will prove itself valuable for engineers designing structural steel buildings in Japan or applying Japanese technology outside of Japan and to fulfill general interest in Japanese design and construction. ■

Fig. 1 Sample Page Excerpted from 2005 AIJ *Design Standard for Steel Structures*

CHAPTER 4 MATERIAL

4.1 Structural Steels and Steel Components

Structural material to be used under this *Standard* shall conform to one of the specifications listed in Table 4.1.

Table 4.1 Specifications and Material Designations

Specification	Title and Material Designation
JIS G 3136	Rolled steels for building structure SN400 A, SN400 B, SN400 C, SN490 B, SN490 C
JIS G 3101	Rolled steels for general structure SS400, SS490, SS540
JIS G 3106	Rolled steels for welded structure SM400 A, B, C, SM490 A, B, C, SM490 YA, YB, SM520 B, C, SM570
JIS G 3114	Hot-rolled atmospheric corrosion resisting steel for welded structure SMA400 A, B, C, SMA490 A, B, C
JIS G 3475	Carbon steel tubes for building structure STKN400 W, B, STKN490 B
JIS G 3444	Carbon steel tubes for general structural purposes STK400, STK490
JIS G 3466	Carbon steel square and rectangular tubes for general structure STKR400, STKR490
JIS G 3138	Rolled steel bars for building structure SNR400 A, B, SNR490 B
JIS G 3350	Light gauge steel sections for general structure SSC400
JIS G 3353	Welded light gauge steel H sections for general structure SWH400
JIS B 1186	Sets of high strength hexagon bolt, hexagon nut and plain washers for friction grip joints
JIS B 1178	Foundation bolts
JIS Z 3211	Covered electrodes for mild steel, high strength steel and low temperature service steel
JIS Z 3351	Solid wires for submerged arc welding of carbon steel and low alloy steel
JIS Z 3352	Fluxes for submerged arc welding
JIS G 5101	Carbon steel castings SC480
JIS G 5102	Steel castings for welded structure SCW410, SCW480

Table 1 Structural Steel Products Applied in Building Construction

Steel type	Designation and grade		F (N/mm²) Thickness (mm) ≦40	>40	Yield-to-tensile strengths ratio, max, %	Plate	Section	Bar	RHS	CHS	Cold formed section
Rolled steels for general structure	SS	400	235	215	-	●	●	●			
		490	325	295	-	●	●	●			
		540	375	-	-	●	●	●			
Rolled steels for welded structure	SM	400	235	215	-	●	●				
		490	325	295	-	●	●				
		520	355	335#	-	●	●				
		570	400	400	-	●	●				
Rolled steels for building structure	SN	400	235	215	80	●	●				
		490	325	295	80	●	●				
Welded light gauge steel H section for general structure	SWH	400	235	215	-		●				
Carbon steel square and rectangular tubes for general structure	STKR	400	235	215	-				●		
		490	325	295	-				●		
Carbon steel tubes for general structure	STK	400	235	215	-					●	
		490	325	295	-					●	
Carbon steel tubes for building structure	STKN	400	235	215	80*					●	
		490	325	295	80*					●	
Rolled steel bars for building structure	SNR	400	235	215	-			●			
		490	325	295	-			●			
Light gauge steel sections for structure	SSC	400	235	215	-						●
Hot-rolled atmospheric corrosion resisting steels for welded structure	SMA	400	235	215	-	●	●				
		490	325	295	-	●	●				

* Max. 85% for arc-welded pipe. # 315 N/mm² for thickness over 75 mm.

Seismic Retrofitting of Long-span Steel Structure Using Viscosity Dampers

by
Hideki Idota
Professor, Nagoya Institute of Technology
Yukimori Yanagawa
KOZO KEIKAKU ENGINEERING Inc.
Hitoshi Niimi
TOYOTA INDUSTSTRIES CORPORATION

Hideki Idota: After finishing the doctor's course of the Tokyo Institute of Technology in 1988, he became assistant professor of the Faculty of Engineering, Nagoya Institute of Technology in 1998. He assumed his current position as professor, the Graduate School of Engineering, Nagoya Institute of Technology in 2009. He received the Architectural Institute of Japan Prize 2015 (Research Themes Division).

Need for Seismic Retrofitting of Buildings in Long Service

In Japan, a number of long-span steel-structure plant buildings that were constructed in the high economic growth period in the 1960s and 1970s are still in use. The Building Standard Law of Japan was revised in 1979 and requires many buildings constructed before 1979 to be seismically retrofitted. However, the seismic retrofitting of these buildings shows no steady progress due to the following reasons:

- To implement the seismic retrofitting that would satisfy the provisions in the current Building Standard Law, it would be necessary to temporarily suspend plant operations, and this would bring about great economic loss.
- While if a plant were moved to another site, the suspension of operations could be avoided, but this would require securing a new plant site and a huge sum of cost for movement.
- Because a clear relation has not yet been established between the parameters (Is: seismic index of structures) applied for the seismic retrofitting design currently in use and the level of damages caused by an earthquake, seismic retrofitting based on performance-based design cannot be applied.

In order to handle these issues, retrofitting by the use of viscosity dampers was successfully applied to a steel-structure large-span building (Figs. 1 and 2, Table 1), an outline of which is introduced in this article.

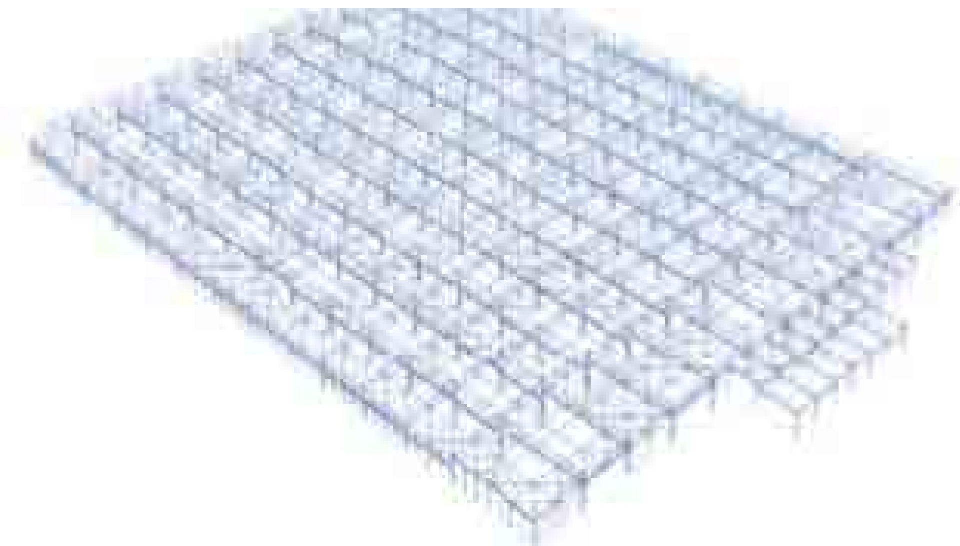
Estimating of Responses under Expected Seismic Waves

In order to examine a method for seismic retrofitting of the target building, dynamic response analysis was conducted using various earthquake mo-

Table 1 Outline of the Plant Building Targeted for Retrofitting

No. of stories	1 story aboveground, no underground floor and penthouse
Total building area	About 33,600 m^2
Plane shape	Long side: 13 spans, about 200 m; Short side: 12 spans, about 160 m
Eaves height, building height	Eaves height: GL=8.2 m; Building height: 11.5m
Year of design	1966 (based on the former Seismic Design Code)
Column	Built-up column using rolled H-shapes
Beam	Truss beam using L-shapes
Roof	Saw-tooth configuration, slate covering
Foundation	Independent foundation, steel pipe pile, exposed column base
Earthquake load	Unit load that reflects latest equipment load: 1.84 kN/m^2
Seismic resistance diagnosis result	Minimum value of seismic index of structure (Is value): 0.17; Average value: 0.30
Natural period (s)	No retrofitting Long side: 1.21; Short side: 0.89
	Extremely rare equivalence Long side: 1.78; Short side: 1.70

Fig. 1 Perspective Drawing of 3D Analysis Model



tions. The earthquake motions adopted for examination are the following three:

- **Wave 1: Simulated Earthquake Motion for Structural Design**

This is ground surface motion amplified by reflecting the results of surveys of the ground at the target building site conducted based on the motion on the engineering bedrock that is provided in the Building Standard Law. The return period of the simulated earthquake motion is 500 years.

- **Wave 2**

This is an earthquake motion obtained by reducing the peak ground velocity of Wave 1 to 80%. The return period of the earthquake motion is 300 years.

- **Wave 3**

This is an earthquake motion expected to occur in the Great Nankai-Trough Earthquake. The occurrence probability for this ground motion is 10% in coming 30 years.

Fig. 3 shows the maximum acceleration response spectrum and the maximum displacement response spectrum of each of these three waves. The figure also shows the primary natural period for the target building obtained by using a 3D analysis frame model.

According to the figure, the natural period of the target building is about 1.1 seconds in the east-west direction, and about 0.9 seconds in the south-north direction. When the natural period is made shorter-cycled by applying seismic retrofitting to this building, the natural period of the target building approaches the predominant period, and thus it is estimated that earthquake motions input into the building rapidly increases.

Under such conditions, in cases when retrofitting is examined that aims at improving the strength of a building, the scale of retrofitting that maintains the building within an elastic range against a response acceleration surpassing 500 (cm/m^2) tends to become large. Accordingly, it was judged not rational for this building to apply a retrofitting method that leads to a shorter natural period.

Estimation of Ultimate Deformation and Settlement of Design Criteria

A conference with the owner pertaining to the criteria in the retrofitting design was held based on the characteristic

Fig. 2 Detailed Configuration of Saw-tooth Roof

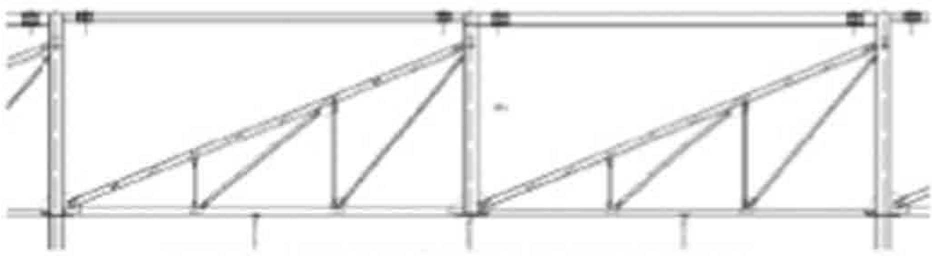
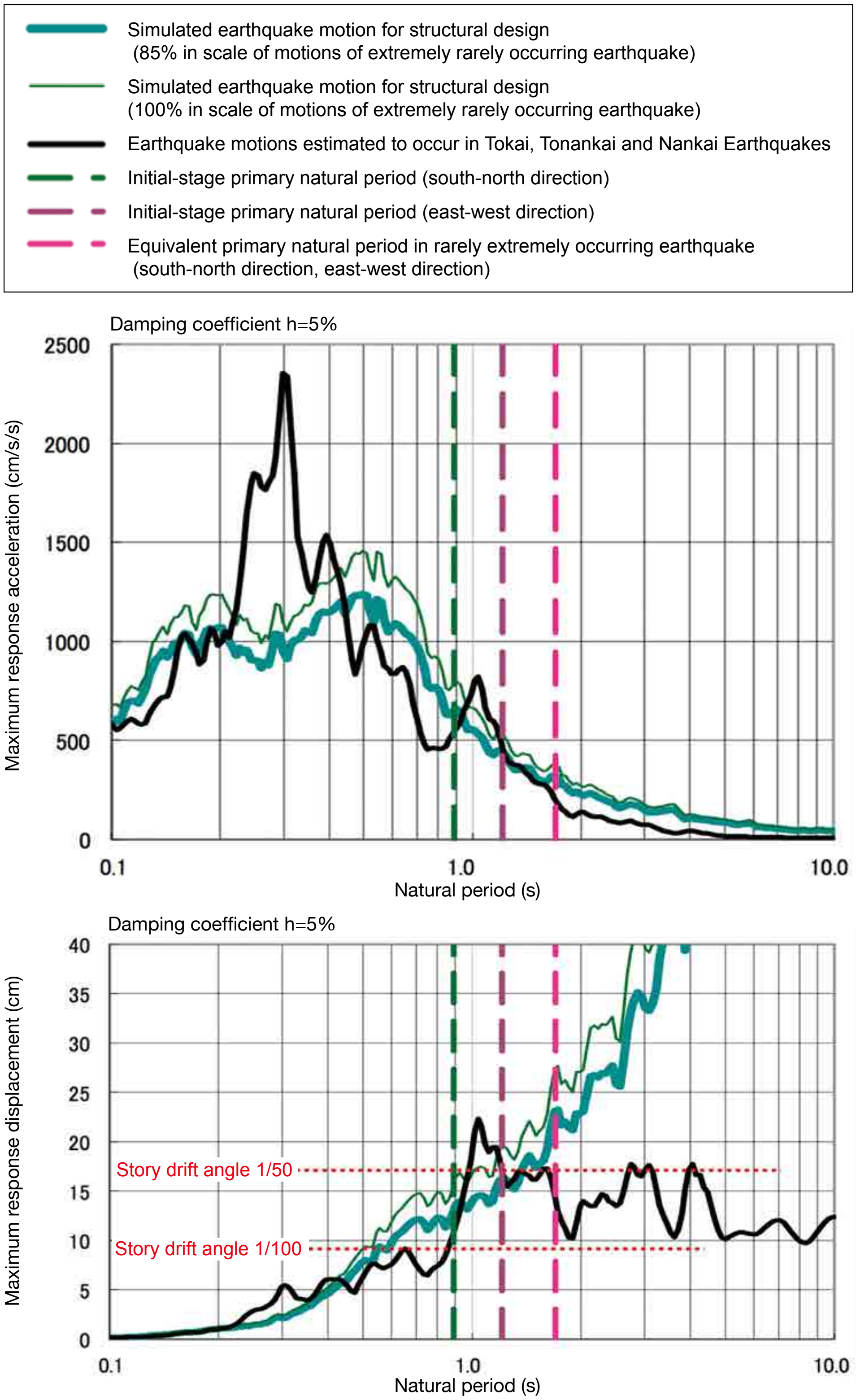


Fig. 3 Maximum Response Spectrum



features peculiar to the target building introduced in the above. The intensions of the owner were as follows:

- **Design criterion 1**

The plant building should not collapse in an earthquake that is extremely rare.

- **Design criterion 2**

Damage to a building by a rarely occurring earthquake is allowed, but continued plant operations must be secured even when subjected to such an earthquake.

- **Design criterion 3**

Plant operations are not to be suspended during retrofitting work.

In criterion 1, because the building is in a limit state that targets the occurrence of building collapse, a push over analysis was conducted that takes into account the occurrence of fractures in truss connections. In the analysis, the restoring force characteristic[4] similar to that in the skeleton curve under the cyclic loading was given to handle buckling. Fig. 4 shows the concept for the analytical method.

Fig. 5 shows the results of push over analysis. In the analysis, the first break point of most framings occurs due to yielding caused by the out-of-plane bending at anchor bolts or the base plate of exposed column bases.

In the long-side direction of framing, the truss connections of most framings fracture when the story drift angle surpasses 1/70, and after lowering the strength of framings by about 15% due to truss connection fracture, the strength of the framings shows no negative gradient up to around 1/50 of the story drift angle even when taking the P-Δ effect into account.

In the short-side direction of framing, while a connection fracture occurs in an early stage in the existing brace of axis 14, strength lowering does not occur up to around 1/40 of the story drift angle in the axes other than axis 14.

Outline of Seismic Retrofitting

Seismic retrofitting by the use of buttress framing was undertaken for the target building taking into account the judgement that, as mentioned earlier, seismic retrofitting by means of strength improvement is not rational for the target building. Specifically, as shown in Fig. 6, steel-structure buttress framing was newly installed on the steel pipe supporting piles driven

Fig. 4 Concept of Push Over Analysis of Phased Strength Loss Type[5]

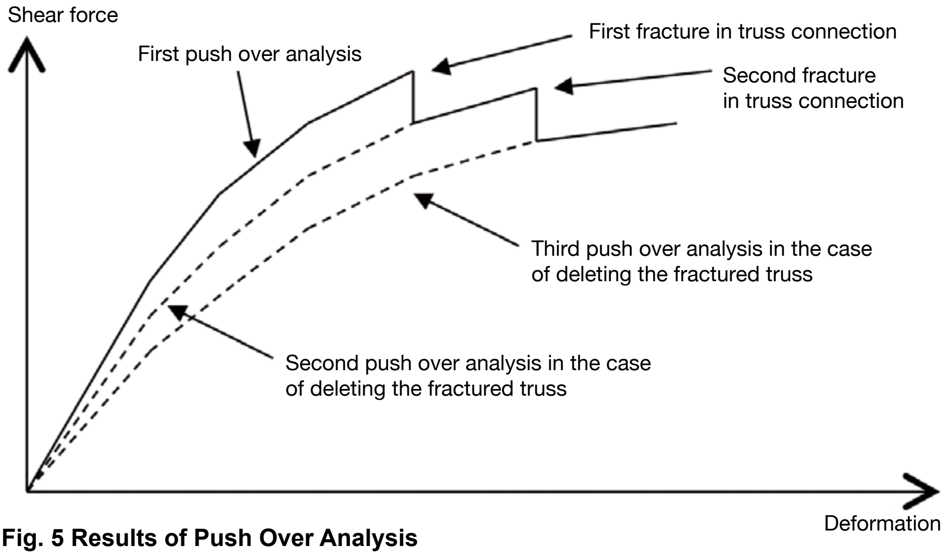


Fig. 5 Results of Push Over Analysis

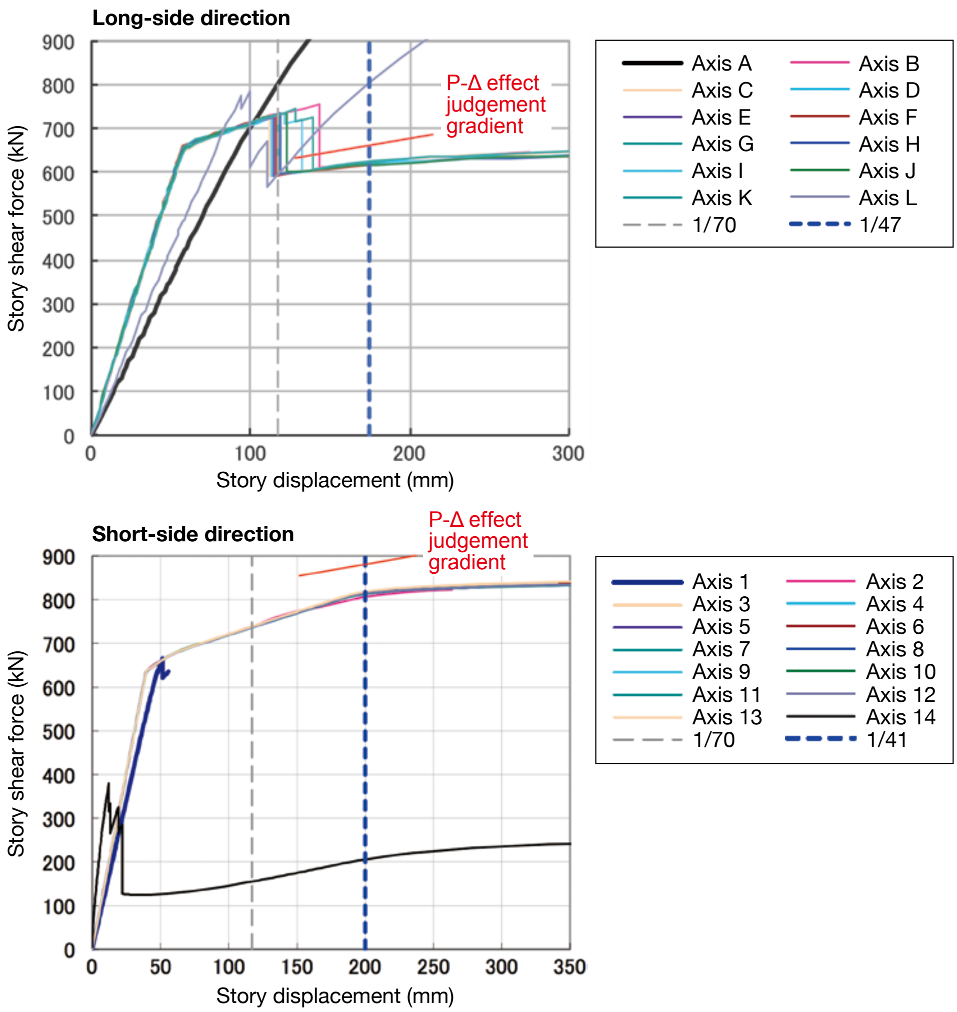
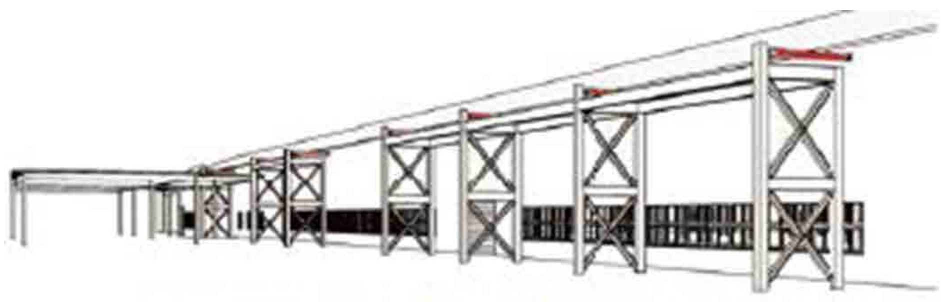


Fig. 6 Response-control Buttress Framing Installed Outside of Plant



outside the building, and the building and the buttress framing were connected using viscosity response-control dampers[7]. The buttress framing was installed on both sides of the structural planes of the building.

Estimating of Maximum Response after Seismic Retrofitting

In order to estimate the maximum response occurring after retrofitting of the target building, a seismic response analysis was made using a 3D model to which the response-control buttress framing was added.

Analytical results show that the maximum response displacement could be set at its specified criteria (within 1/70 of the story drift angle) when subjected to earthquake motions in waves 2 and 3 mentioned earlier. In the case when subjected to the earthquake motion in wave 1, while the story drift angle in which the maximum response displacement surpasses its specified criterion occurs, it was confirmed that building collapse and beam falling caused by the fracture of chord members do not occur. The fracture occurrence in chord members discussed here was judged based on the fracture occurrence criterion in which the axial strain at the tension side does not surpass 1% as shown in Fig. 7.

Attainments of Retrofitting

The following useful results and future tasks were obtained in the current retrofitting project:

- Seismic retrofitting that satisfies the building performance desired by the owner was realized with extreme economy by allowing the buckling of chord members.
- In the seismic retrofitting of steel-structure large-span buildings that require sustained business operations, it will be necessary to develop a seismic retrofitting technology that effectively improves seismic resistance by retrofitting only from outside the buildings.
- It is important to clarify the relation between the scale of earthquake motion input for retrofitting design and the resulting damage estimated to occur, and to devise technology for easily explaining such a relation to owners.

■

Fig. 7 Hysteresis of Axial Strain-Axial Stress of the Truss that Causes Buckling

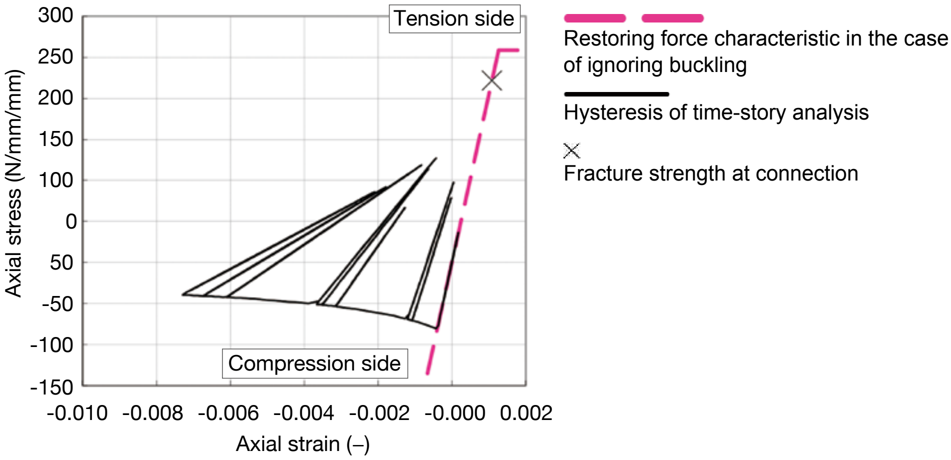


Photo 1 Response-control buttress framing employing viscosity damper

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Guidebook for Preventing Brittle Fractures of Inner Diaphragm Electro-slag Welds
—For Use in the Manufacture of Built-up Box Section Columns—

by Takahiko Suzuki, Nippon Steel & Sumikin Technology Co., Ltd., and Takumi Ishii, JFE Techno-Research Corporation

Increasing Need for Preparation of Guidebook

The Japanese Society of Steel Construction published in 2016 the *Guidebook for Preventing Brittle Fractures of Inner Diaphragm Electro-slag Welds*[1)] for use in the manufacture of built-up box section columns. The research for the preparation of the *Guidebook* was commissioned by the Japan Iron and Steel Federation.

Built-up box section columns are manufactured by weld-assembling four steel plates (hereinafter referred to as “box columns”). Box columns have been extensively adopted for construction of the low-rise stories of high-rise office buildings, and the thickness of the steel plates used ranges from 25 mm to 100 mm. The standard welding practice adopted for joining the inner diaphragms of box columns is electro-slag welding, and there are cases in which the maximum heat input in welding reaches 1,300 kJ/cm. (Refer to Fig. 1)

It is believed that the toughness of the column skin plate is greatly lowered due to such a large heat input and stress is concentrated on the slit (Fig. 2) between the backing metal of the electro-slag weld (hereinafter referred to as “ESW”) and the skin plate of the box column, and as a result brittle fracture occurs in the ESW. While there are no examples of fractures occurring in steel frames damaged in past earthquakes, fractures occurring in ESW (Fig. 3) have been found in research being promoted on the fracture of column-bean connections[2)].

In the Hyogoken-Nanbu Earthquake (Great Hanshin Earthquake) of 1995, fractures occurred in the CO_2 weld connections of steel-frame beam ends, and as a countermeasure against such fractures, the *Guidelines for Preventing Brittle Fracture in Beam-end Welded Connections* was published by the Building Center of Japan[3)]. In the *Guidelines*, steel-frame beam ends are required to possess a toughness of 70J or higher

Fig. 1 Manufacture of Built-up Box Section Column (Box Column) by Means of Large Heat-input Welding

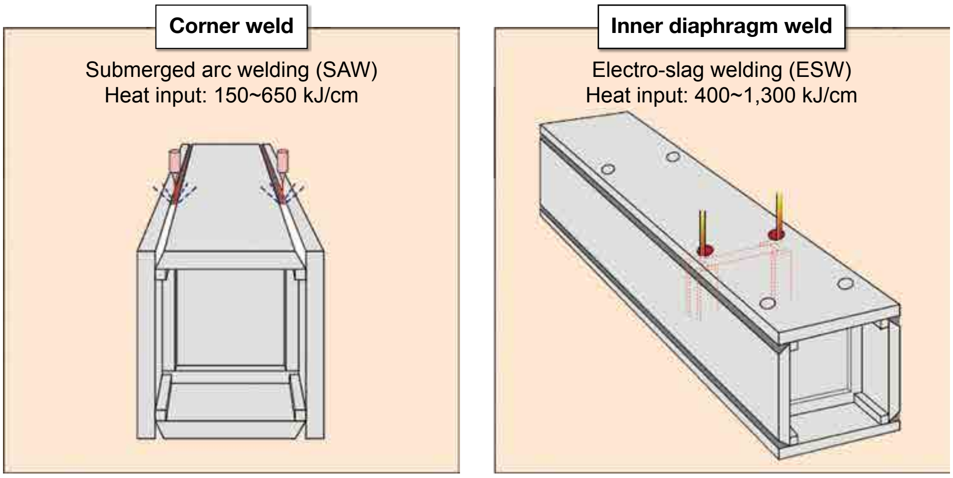


Fig. 2 Outline of Column-Beam Connection of Box Column and Anxiety over Brittle Fracture Occurrence

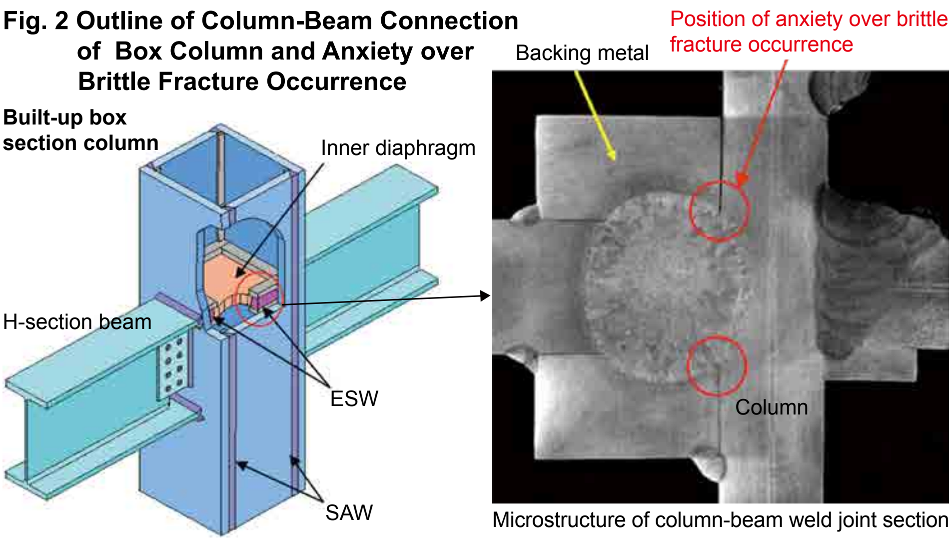
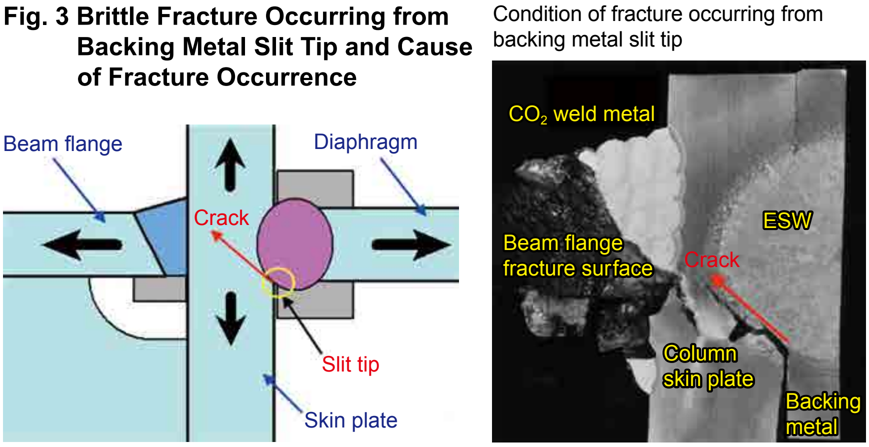


Fig. 3 Brittle Fracture Occurring from Backing Metal Slit Tip and Cause of Fracture Occurrence



(70J: Charpy absorbed energy at a test temperature of 0°C). However, because welding heat that is several tens of times more than that in CO_2 welding is input during electro-slag welding, it is difficult to secure a toughness of 70J for ESW.

To cope with such a situation, high-performance steel has been developed in which the lowering of toughness in large heat-input welding is suppressed (high HAZ toughness steel[4]). However, because the plate thickness of column skin plate tends to be thinner due to the recent wider application of concrete-filled steel tube columns, it has become difficult to secure an appropriate toughness of ESW even with the use of high HAZ toughness steel. Meanwhile, when electro-slag welding is replaced with CO_2 welding, steel-frame productivity is greatly reduced.

Given these circumstances, it has been required to discover a means to prevent the occurrence of brittle fracture that takes into full account the currently-applied steel products, welding materials and weld execution conditions as well.

The Japan Iron and Steel Federation conducted structural testing (Fig. 4) that adopts as parameters the toughness of welds and the tensile stress of column skin plate to reproduce the conditions for ESW fracture (Fig. 5). The relation between the ESW toughness and the ESW fracture strength was found from the test results (Fig. 6). Further, the local stress at the fracture initiation point (equivalent maximum main stress) was found by means of FEM analysis (Fig. 7) thereby confirming the relation between the ESW toughness level (Charpy impact value) and the upper-limit stress (Fig. 8). A series of the results obtained from the above (for example, the reference 5) are organized in the current *Guidebook for Preventing Brittle Fractures of Inner Diaphragm Electro-slag Welds*.

Fig. 4 Understanding of Fracture Occurrence Condition by Means of Structural Testing

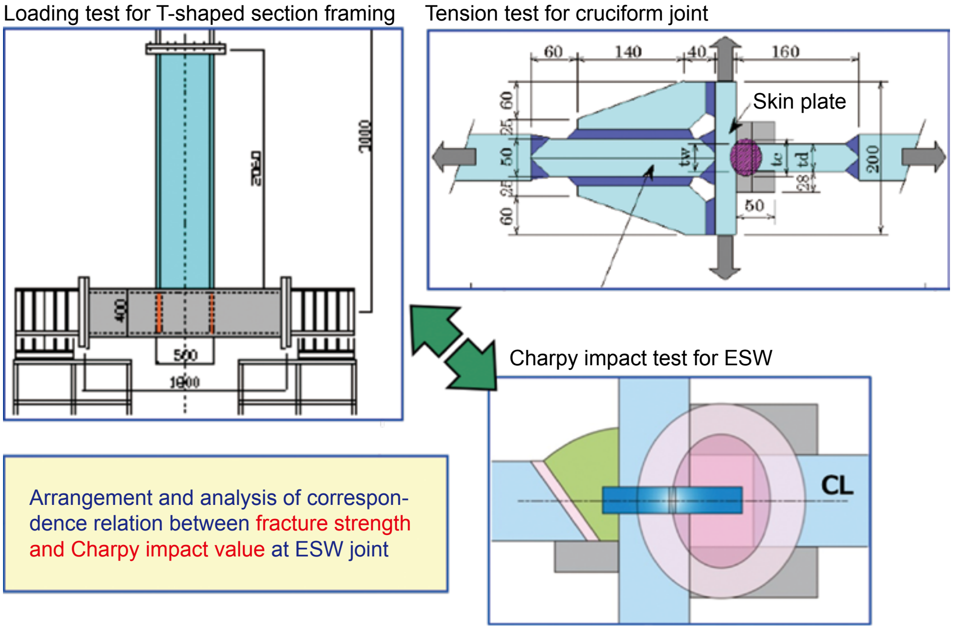


Fig. 5 Partial Framing Test: Example of Fracture Condition

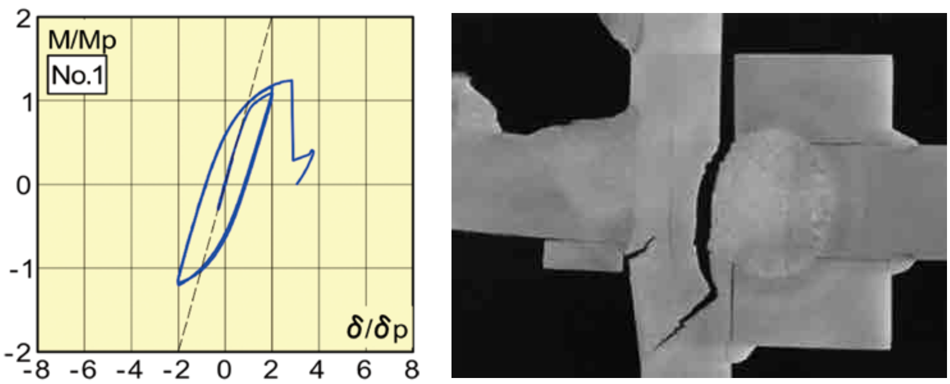
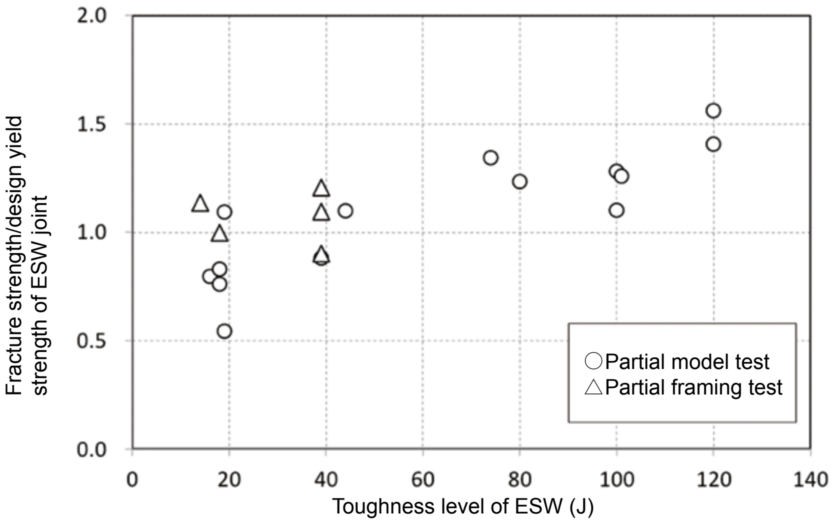


Fig. 6 Relation between Fracture Strength and Toughness Level of ESW Joint



Outline of the Guidebook

According to the *Guidebook for Preventing Brittle Fractures of Inner Diaphragm Electro-slag Welds*, the tensile force (tensile stress) working on ESW is suppressed to lower levels based on the toughness of practical ESW, capitalizing on which ESW fractures are prevented from occurring. This is because both the fracture strength (upper-limit stress) of ESW and the toughness of ESW simultaneously lower, as shown in Figs. 7 and 8. In the following, the methods to prevent ESW fractures from occurring are introduced:

• Method by Means of Easy Examinations of Fracture Prevention

Fig. 9 shows the flow of easy examinations required for ESW fracture prevention. As seen in the flow, at first the toughness level of ESW is determined, and then fractures are prevented from occurring by setting the macroscopic stress working on the diaphragm at the level lower than the upper-limit value for applied stress shown in Table 1. The applied stress $_{d}\sigma$ is calculated using the following equation.

$$_{d}\sigma = \frac{_{cf}M}{(_{d}t+\Delta t)\cdot(_{b}H+_{bf}t)\cdot(_{b}B+2_{s}t)}$$

$_{cf}M$: Bending moment working on beam end (column face) (N•mm)
$_{d}t$: Thickness of inner diaphragm (mm)
Δt: Penetration width (mm) *Sum of penetration width at both sides
$_{b}H$: Beam height (mm)
$_{bf}t$: Beam flange thickness (mm)
$_{b}B$: Beam flange width (mm)
$_{s}t$: Thickness of column skin plate (mm)

In the calculation of the applied stress $_{d}\sigma$, the standard toughness of ESW is set at 27J. However, in cases when the applied stress $_{d}\sigma$ surpasses 240 N/mm^2 or in cases when the standard toughness 27J cannot be secured, the upper-limit applied stresses corresponding to the toughness levels $_{v}E$=15J and $_{v}E$=47J, which are shown in the table, are applied.

Fig. 7 Understanding of Fracture Occurrence Condition by Means of FEM Analysis

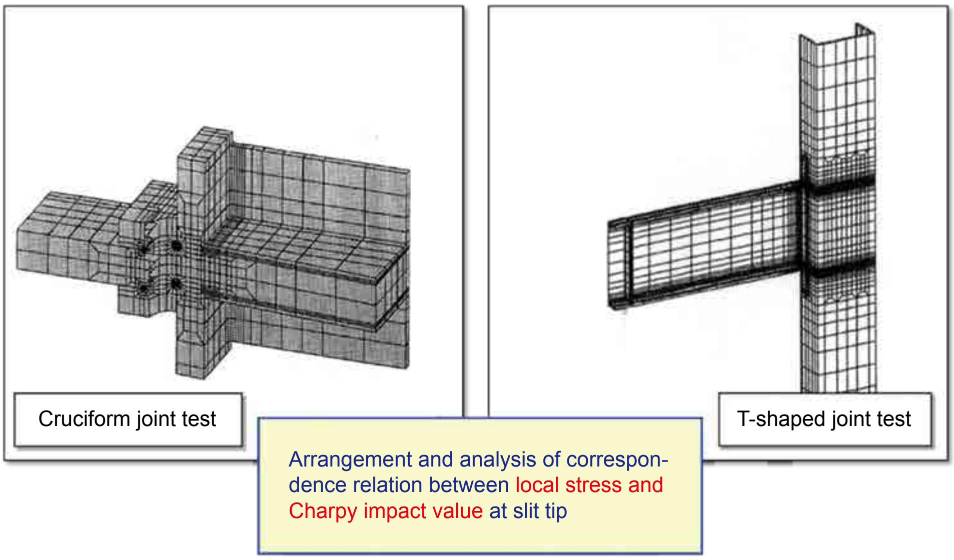


Fig. 8 Relation between Equivalent Maximum Main Stress at Fracture Initiation Point and Toughness Level

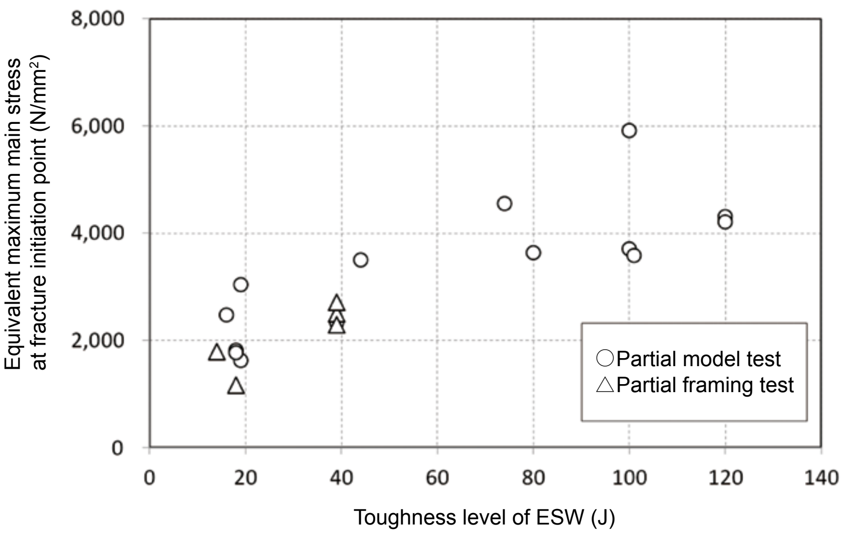


Table 1 Standard for Upper-limit Value of Applied Stress of Inner Diaphragm

Toughness level of ESW ($_{v}E$)	15J or higher	27J or higher	47J or higher
Standard for upper-limit value for applied stress ($_{d}\sigma$)	160 N/mm^2 or lower (0.5×F)	240 N/mm^2 or lower (0.75×F)	325 N/mm^2 or lower (1.0×F)

• Method by Means of Detailed Examinations of Fracture Prevention

In the method by means of detailed examinations of fracture prevention, fracture prevention is examined based on the local stress occurring in the slit between the backing metal and the column skin plate of ESW. This method takes into account the tensile stress of the column skin plate, and thus a more rational fracture prevention design becomes available compared to the fracture prevention design using the easy examination method mentioned above.

Fig. 10 shows the flow of detailed examinations required for ESW fracture prevention. The examination flow consists of three procedures: 1) calculation of the stress occurring in the inner diaphragm, 2) calculation of the maximum main stress working on the fracture initiation point, and 3) determination of the toughness required for avoiding the occurrence of brittle fracture. The required strength γ_{req} is calculated from the design conditions employing the procedures 1), 2) and 3), and the calculated value is compared with the upper-limit strength γ_{lim} calculated from the toughness of ESW. In cases when the comparison result is $\gamma_{req} \leqq \gamma_{lim}$, the examinations end, and in cases when comparison result is $\gamma_{req} > \gamma_{lim}$, the connection detail and the toughness shown in the design conditions are reexamined.

• Assessment Method for the Toughness of ESW

The toughness of ESW is assessed by means of impact tests in the weld execution tests that are conducted using actual steel products and welding materials and under practical welding conditions. It is known that the toughness of ESW greatly differs depending on the notch position of the test specimens. While a uniform procedure for the extraction and preparation of test pieces has thus far not been adopted, the *Guidebook* has prescribed such a test procedure shown below.

According to the *Guidebook*, the test specimen is extracted from the position shown in Fig. 11, and the impact test is implemented in three different notch positions. This is because the initiation point of the fracture is the fusion zone of

Fig. 9 Flow of Easy Examinations Required for ESW Fracture Prevention

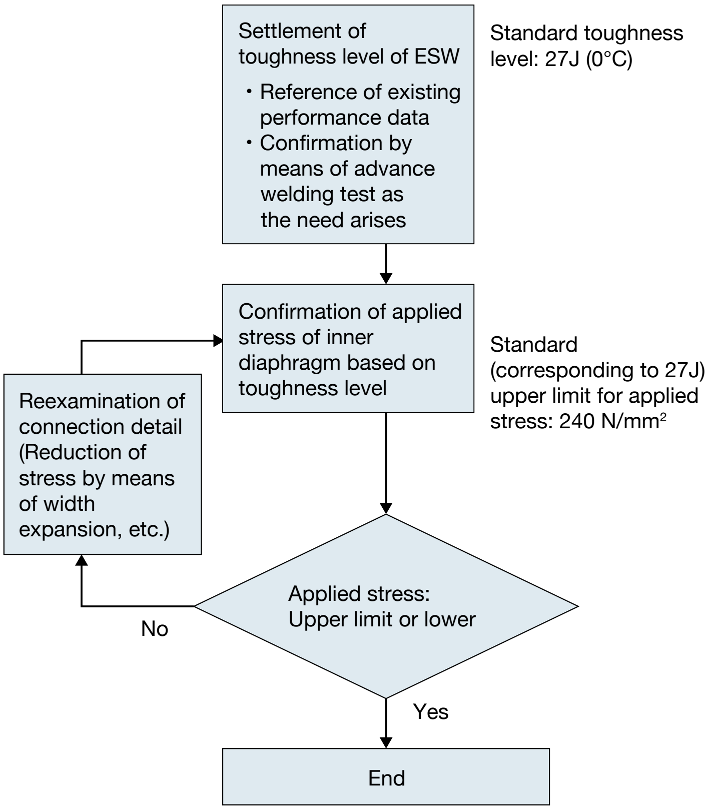
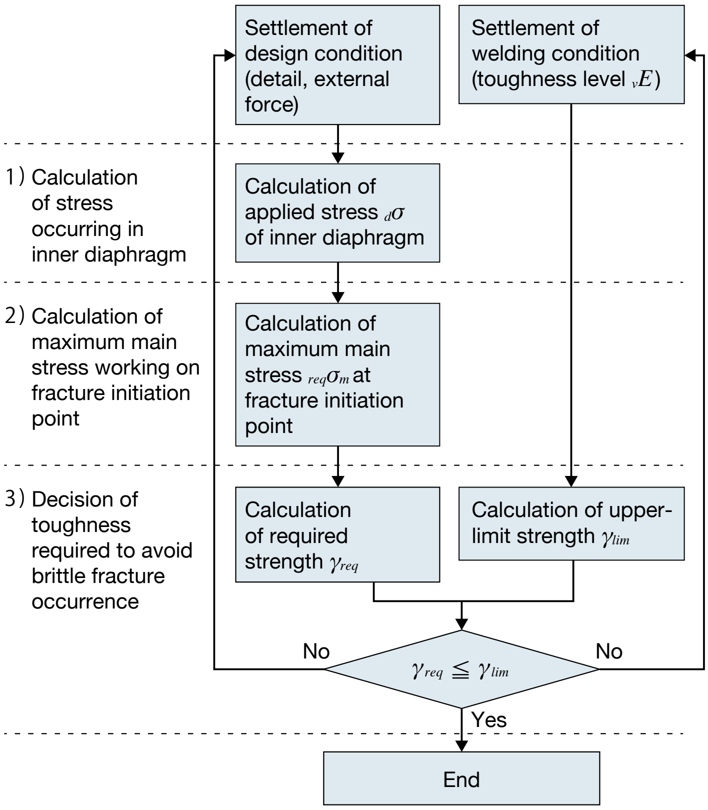


Fig. 10 Flow of Detailed Examinations Required for ESW Fracture Prevention

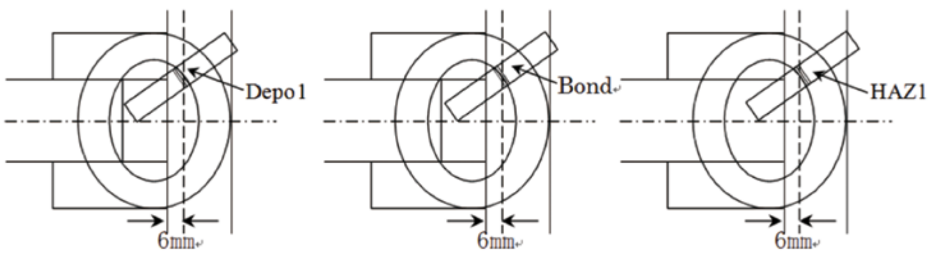


ESW at the slit tip, and the fracture occurs in the weld metal or the heat-affected zone (HAZ) of ESW. Specifically, the three notch positions are:

- Bond: Fusion zone of the column skin plate and the weld metal
- HAZ1: A position 1 mm from the bond section to the column skin plate side
- Depo1: A position 1 mm from the bond section to the weld metal side

Three impact tests are conducted at each of the three notch positions to find the average value for Charpy absorbed energy. Of the average values thus found, the lowest value is set at the toughness of ESW. The test temperature to be basically adopted for the impact test is 0°C, but in cases when the steel-frame application environments and conditions greatly differ one from the other, the test temperature is changed.

Fig. 11 Procedure for Extraction and Preparation of Test Specimens



Future Plans pertaining to the Guidebook

The current *Guidebook* targets the ESW of 490 N/mm^2-grade steel. Meanwhile, recent trends show that 590 N/mm^2-grade steel is increasingly being adopted for the construction of buildings that are tending to become gigantic. To that end, it is considered necessary to make examinations similar to those mentioned above in the application of high-strength steel with tensile strength ratings of 490 N/mm^2 or more. ■

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3) The Building Center of Japan: Guidelines for Preventing Brittle Fracture of Beam-end Welded Connections, Sept. 2003 (Related article: Design to Prevent Brittle Fracture in Beam-end Connections, SCT&T No. 6)

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Latest Information about Steel Products for Building Structures in Japan

Committee on Overseas Market Promotion, The Japan Iron and Steel Federation

The Japan Iron and Steel Federation (JISF), jointly with steelmakers who are JISF member companies, is promoting research on advanced product standards and application technologies pertaining to structural steel products developed in Japan. This article introduces information on the recent results of joint research on the standardization of structural steel products and the revision of guidelines regarding four steel products destined for building construction.

Publication of Guidelines for Application Technologies for H-SA700 (780 N/mm^2-grade High-strength Steel for Building Structures)

H-SA700 is a steel product that was certified by the Ministry of Land, Infrastructure, Transport and Tourism in 2012 and marketed as the product having a standard common for steelmakers. Two designations have been adopted for the steel: H-SA700A for use for non-welding and H-SA700B for use for welding. (Refer to Table 1)

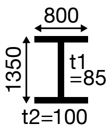
The *Guidelines for Welding of H-SA700* prepared by JISF has thus far been available as a technical document for the application of H-SA700. In March 2017, JISF issued the *Guidelines for Application Technologies for H-SA700*. The new *Guidelines* cover not only the conventional *Guidelines for Welding* but the *Design Guidelines for H-SA700* (draft), which incorporate the achievements attained in joint research with related organizations. The new *Guidelines* are compiled as a technical document relating to both the design and application of H-SA700. (Refer to Fig. 1)

Table 1 Features of H-SA700 in Mechanical Properties

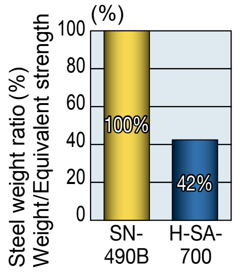
Designation	Thickness (mm)	Yield strength (N/mm²)	Tensile strength (N/mm²)	Yield ratio (%)	Impact energy (J)	Ceq (%)	P_{CM} (%)
SN490B	6≦t<12	325≦	490～610	–	[0℃] 27≦	(t≦40) ≦0.44 (40<t) ≦0.46	≦0.29
	12≦t<40	325～445		≦80			
	40≦t≦100	295～415					
H-SA700A	6≦t≦50	700～900	780～1000	≦98	[0℃] 47≦	≦0.65	≦0.32
H-SA700B					[-20℃] 47≦	≦0.60	≦0.30

*SN490B: Conventional steel product for building construction

Fig. 1 Member Downsizing Attained by the Use of H-SA700, and Resulting Steel Weight Reduction



Dimension	SN490B	H-SA700
Fy (yield strength) (N/mm²)	295	700
Web thickness t1 (mm)	85	40
Flange thickness t2 (mm)	100	35
Section area (cm²)	2578	1085



Revision of Guidelines for the Design and Welding of SA440 (590 N/mm^2-grade High-performance Steel for Building Structures)

SA440 is a steel product that is not only high in tensile strength (590~740 N/mm^2) but low in yield ratio (80% or lower), and, further, features less deviation in mechanical properties. Its chemical composition is designed so that the weld-crack sensitivity is suppressed to a minimum, and its weldability is greatly improved over conventional 600 N/mm^2-grade steel. (Refer to Table 2) To that end, it is a steel product that is easy to apply in terms of both design and construction. Design by using SA440 high-strength steel facilitates a remarkable reduction in not only the sectional dimensions and the weight of structural members applied but also the fabrication work needed. (Refer to Fig. 2)

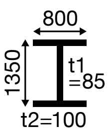
JISF issued in October 1996 the first edition of *Guidelines for Design and Application of High-performance 590 N/mm^2-grade Steel for Building Structures (SA440)*. It was revised in August 2004 to incorporate new information such as measures to respond to the revision of both the Building Standard Law of Japan and JIS specifications for welding materials in June 2000, straightening technology and stud welding.

The current revision of the *Guidelines* made in March 2016 has two purposes: reflection on the *Guidelines*, the establishment and revision of related laws and ordinances and the revision of JIS specifications for welding materials made after 2000; and the addition to the *Guidelines* of amendments and postscripts in order to reflect the application of large heat-input welding for built-up square columns and other new technological knowledge and information about accumulated results of the application of SA440 thus far attained.

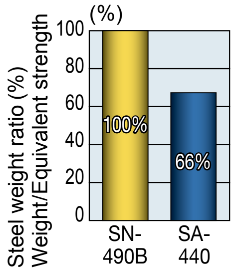
Table 2 Features of SA440 in Mechanical Properties

Designation	Thickness (mm)	Yield strength (N/mm²)	Tensile strength (N/mm²)	Yield ratio (%)	Impact energy (J)	Ceq (%)	P_{CM} (%)
SN490B	6≦t<12	325≦	490～610	–	[0℃] 27≦	(t≦40) ≦0.44 (40<t) ≦0.46	≦0.29
	12≦t<40	325～445		≦80			
	40≦t≦100	293～415					
SA440C	19≦t≦100	440～540	590～740	≦80	[0℃] 47≦	(t≦40) ≦0.44 (40<t) ≦0.47	≦0.28 ≦0.30

Fig. 2 Member Downsizing Attained by the Use of SA440, and Resulting Steel Weight Reduction



Dimension	SN490B	SA440
Fy (yield strength) (N/mm²)	295	440
Web thickness t1 (mm)	85	65
Flange thickness t2 (mm)	100	50
Section area (cm²)	2578	1711



Establishment of Standards MDCR0016 and 0017-2016 for TMCP Steel Products for Building Structures

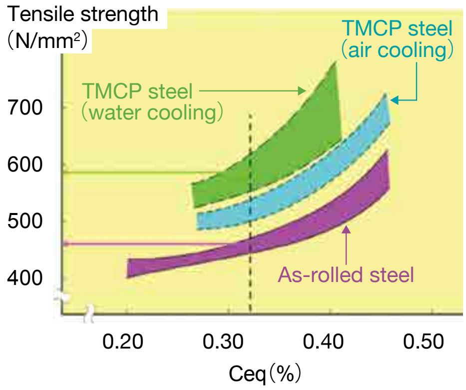
TMCP (thermo-mechanical control process) steel products for building structures have been developed to meet the increasing size of framing member while buildings are getting higher and larger. The product was certified by the Ministry of Land, Infrastructure, Transport and Tourism for marketing and has been extensively applied in the field of building construction.

JISF has established its own standard for TMCP steel products for building structures with the aim of further promoting their application by the following means: setting of common designations and specifications, enhancement of application technologies and promotion of R&D conducive to improving the competitiveness of steel-frame manufacturing technologies. (Refer to Table 3 and Fig. 3)

Table 3 Features of TMCP Steel in Mechanical Properties

Designation	Thickness (mm)	Yield strength (N/mm²)	Tensile strength (N/mm²)	Yield ratio (%)	Impact energy (J)	Weldability Ceq (%)
SN490B	6≦t＜12	325≦	490～610	–	[0℃] 27≦	(t≦40) ＜0.44 (40<t) ＜0.46
	12≦t＜40	325～445		≦80		
	40≦t≦100	295～415				
TMCP325B (MDCR0016)	40≦t≦100	325～445	490～610	≦80	[0℃] 27≦	(t≦50) ＜0.38 (50<t) ＜0.40
TMCP355B (MDCR0016)	40≦t≦100	355～475	520～640	≦80	[0℃] 27≦	(t≦50) ＜0.40 (50<t) ＜0.42
TMCP385B (MDCR0017)	19≦t≦100	385～505	550～670	≦80	[0℃] 70≦	(t≦50) ＜0.40 (50<t) ＜0.42

Fig. 3 High Strength and Good Weldability Offered by TMCP Steel



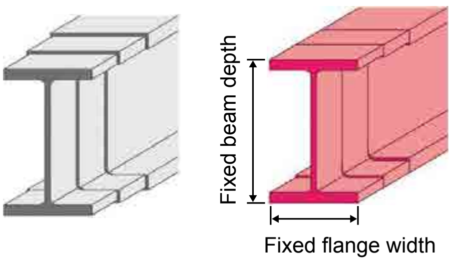
JIS Standardization of H-beams with Fixed Outer Dimensions

JIS (Japanese Industrial Standards) relating to H-beams was revised in 2014, with the new incorporation of standard dimensions for H-beams with fixed outer dimensions. H-beams with fixed outer dimensions are a kind of H-beam having fixed beam depth and fixed flange width in an identical size series (Fig. 4). Standard dimensions incorporated in JIS are:

- Web height: 400~1,000 mm
- Flange width: 200~400 mm

Fixed beam depth and fixed flange width in an identical size series enable simple design and fabrication. The application of H-beams with fixed outer dimensions offers two practical advantages—reduction of the number of filler plates to be applied in bracket-beam bolt connections, made available by making the beam height uniform; and reduction of the number of stiffeners to be installed on the columns in column-beam connections, made available by making the height of the beam to be joined with the column uniform (Fig. 5). These application advantages lead to not only improved connection fabrication efficiency but also to the simple design of entire steel-frame buildings.

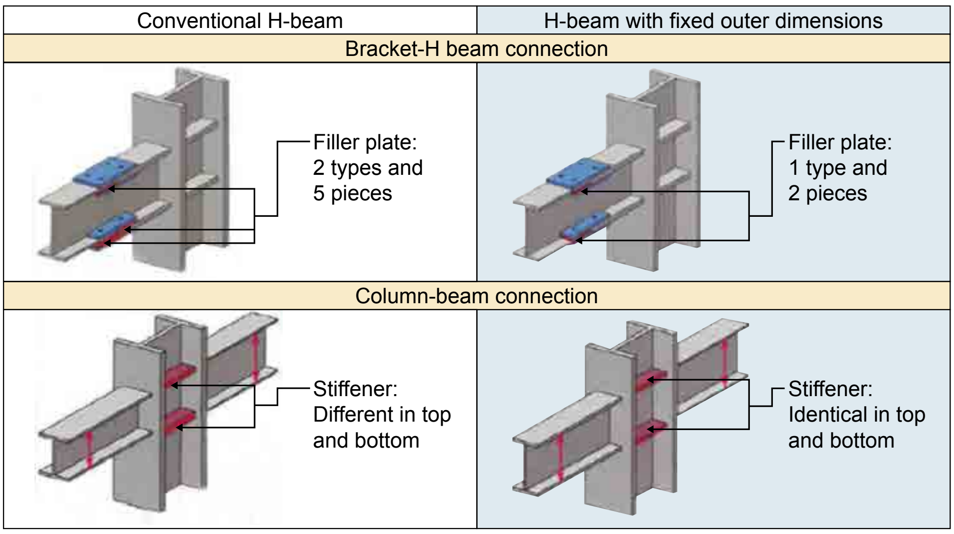
Fig. 4 Comparison between Conventional H-beam and H-beam with Fixed Outer Dimensions



Conventional H-beam (Fixed inner dimension)

H-beam with fixed outer dimensions

Fig. 5 Optimal Structural Design and Construction Cost Savings Attained by the Use of H-beam with Fixed Outer Dimensions



ROKI Global Innovation Center

—Bright, Vast Space Design Employing Steel Frame-Wood Hybrid Trusses—

Tetsuo Kobori Architects and Arup

The ROKI Global Innovation Center (ROGIC) is an R&D building of ROKI Co., Ltd., a global company with advanced filtration technology. What ROKI required of this project was that the "site encouraged creativity" for further development of filtration technology. The huge airy grid-patterned roof, that is made by steel frame trusses restrained buckling by wood members, made it possible to open the work space upon natural light (Photo 1).

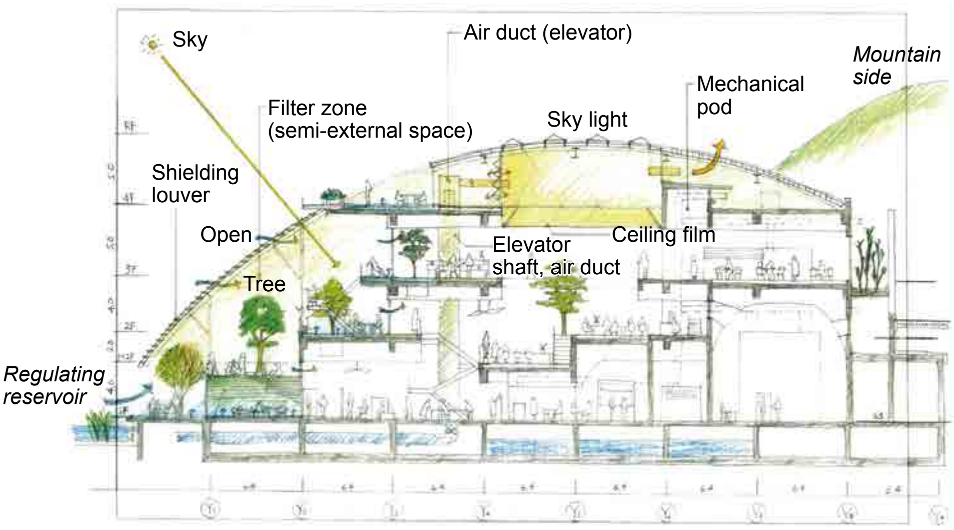
Photo 1 Full view of ROKI Global Innovation Center

Spatial Imaging Makes Optimum Use of Building Site Abundant with Nature

The building site is located in a mountainous area with the Tenryu River flowing below. The site consists of tiered level ground that was left intact after residential land development carried out 30 years ago, and a regulating reservoir is located there as it is hidden from the surrounding area. It is also blessed with a wild environment awash with waterfowl and other inhabitants.

Strongly inspired by these site conditions, we imagined a large single room-like space that integrates the natural environment and the architecture—a spacious area that is brought about by making the most of the topographical conditions that overlook the reservoir and in which the building floors are located as they rise from the ground (Fig. 1 and Photo 2).

Fig. 1 Concept Sketch for ROGIC Building Proposed at Initial Design Stage



Spacious Single Room-like Area with Overlapping Floors

ROGIC has four floors that, capitalizing on the topographical conditions peculiar to the site, are linked to each other in tiers from the south-side reservoir to realize a spacious single room-like space. The building measures 64 m in the east-west direction and 54 m in the south-north direction. The total floor area is 9,000 m^2, where work spaces and laboratory rooms are laid out, in addition to a coffeehouse and terrace, and accommodating around 150 workers.

The main entrance is located on the north side of the fourth floor (mountain side). The first thing that you encounter when entering the ROGIC building is a view overlooking the entire work space and the regulating reservoir on the south side. From this perspective, everyone in the room sees and be seen by everyone else, thus making this perspective useful for researchers.

Photo 2 Peripheral area of ROGIC where the river flows and trees grow thick

Photo 3 Entire working space seen from 4th floor

The building interior is structured so that researchers and workers go down from the entrance toward the lower working spaces. The tiered floors from the second to the fourth are for work spaces, while the first floor contains the experimental area on the north side and the terrace facing the reservoir on the south (Photo 3).

Because we intended to provide a semi-outdoor space in which the indoors feel as if it were outdoors, we planned the entire roof to be composed of a structure like a thin film that is floating lightly in the air. Further, we chose not to install columns within the indoor space, and as a result, the roof, composed of wood-steel hybrid double-skin trusses, was made to cover the entire building structure.

The south side of the roof is entirely composed of glass, and a slit sky light is arranged at the top of the folded-plate roof. A filter produced by ROKI is spread as the ceiling material beneath the lower chord members of the roof. The filter is a non-woven fabric that at first glance seems like Japanese paper. The filter has a sound-absorbing performance, and at the same time diffuses the sunlight so that lighting is not required during the daytime in the indoor space. When it is light outside, the indoor space is light due to the filter performance, and when it gets dark outside, it becomes dark inside. That is, the filter is designed so that it reflects on the indoor space the climatic changes and cloud movements just as they are.

Structural Plan for Space and Equipment

A wood-steel frame hybrid structure is adopted for the construction of the huge roof, and steel-reinforced concrete structures for the other building structures. Taking into account the characteristics of the space and the connections between heavy equipment and devices, the structural type was decided based on the concept of suitable materials for the right places.

The Building Standard Law of Japan specifies the adoption of fireproof construction in the case of wooden buildings having a total floor area of more than 3,000 m^2 regardless of region. However, in the case of a steel-structure building, it specifies the adoption of quasi-fireproof construction depending on the region and does not specify the adoption of fireproof construction for the roof structure. Therefore, in the ROGIC project, we interpreted these specifications to mean that the wooden members served just as finishing members that restrain buckling of the steel-frame lower chord members, and as a result the wooden members were omitted from the list of structural members used for the main structural sections.

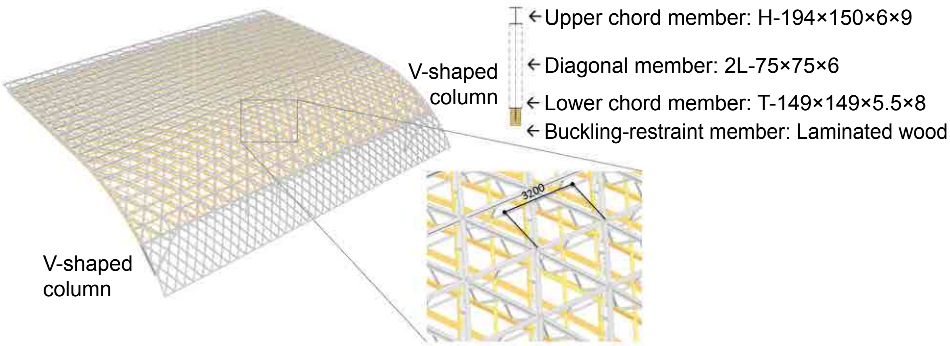
Structuring of a Roof Free Curvature by the Use of Wood-Steel Hybrid Trusses

The huge roof is a wood-steel frame hybrid structure. The beams are installed in two directions to the roof's free curvature at a pitch of 3.2 m.

H-shapes (H-194×150×6×9; SN490) are used as the upper chord members. T-shapes (T-149×149×5.5×8; SN490) are used as the main members of the lower chords. Built-up H-shapes are used for the structural sections in the periphery of the roof supports where stress is concentrated. The wooden members are attached to the lower chord steel frame in order to restrain buckling of the lower chord steel frame.

The lower chord members are installed at a pitch of 3.2 m, which however cannot offer sufficient structural

Fig. 2 Structure of Huge Roof Employing Steel Frame-Wood Hybrid Members



strength, and thus a sub-frame that stiffens the lower chord is additionally installed at a pitch of 3.2 m. T-shapes (T-100×100×5.5×8; SS400) are used as the sub-frame members, and wooden members are likewise used as members that restrain buckling of the sub-frame members. To that end, the lower chord surface of the roof naturally forms a 1.6 m wooden grid due to the application of the lower chord members and sub-frame members in the roof truss. (Refer to Fig. 2)

Because the roof configuration has a two-direction free curvature, configurational twisting and bending occur in the truss beams that span the roof. The roof configuration is symmetrical, and thus two identical members are naturally used for the roof.

As regards the H-shapes used as upper chord members, the flange plate at 6-junction connections is gently folded to match the specified roof curvature. As regards the web plate, tolerance of the web is absorbed by the use of round bars (60 mm dia.) arranged at the center of the connections.

For the connection of the T-shapes used as lower chord member, 7 mm thick diaphragm plates and 6 mm thick web plates, having more thickness than the lower chord members, are adopted to absorb the tolerance occurring in the flanges and webs between each member due to the twisting. Bolt-joining was adopted for the upper chord members to secure higher installation accuracy. Because wooden members are inserted into the lower chord members to restrain buckling, weld-joining was adopted for the lower chord members. (Refer to Fig. 3)

Steel-frame manufacturing accuracy for the truss beams and accuracy in their installation are linked directly to the completion accuracy for the huge roof. Shop manufacturing accuracy for the steel frames was confirmed at the stage of assembly inspection, where workers were engaged in temporary assembly while at the same time securing due manufacturing accuracy for the plates, one by one, using jigs and taking a long time.

Fig. 3 Detail of Steel Frames Used for Roof Truss

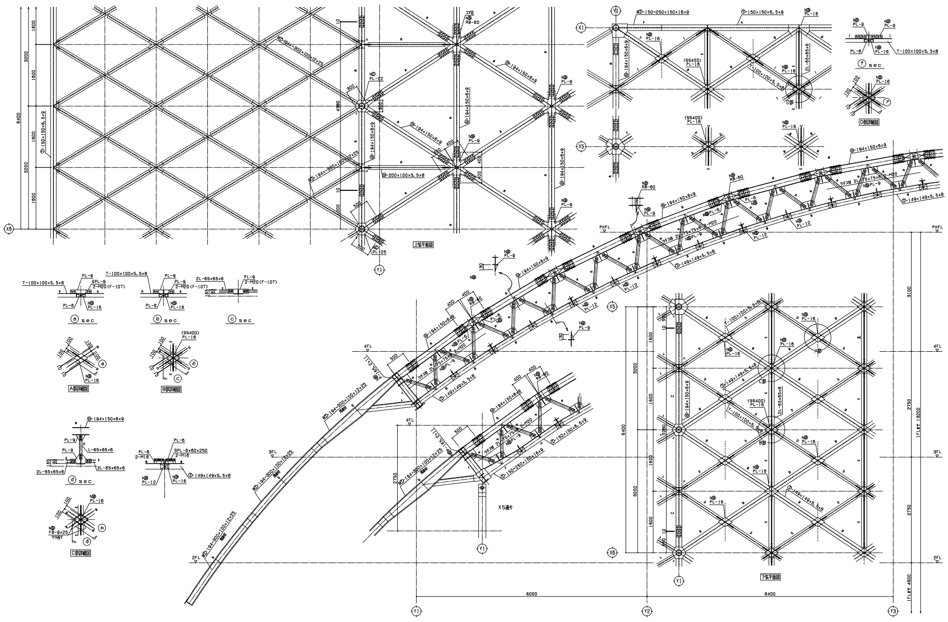


Photo 4 Hoisting and welding of field-assembled roof truss beam units

Photo 5 Fixing wood members to lower chord member

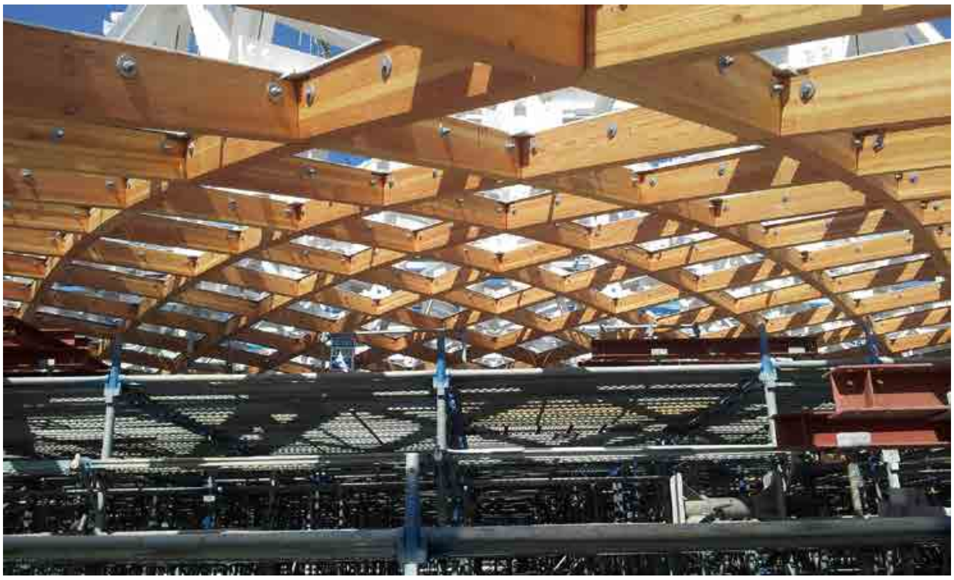


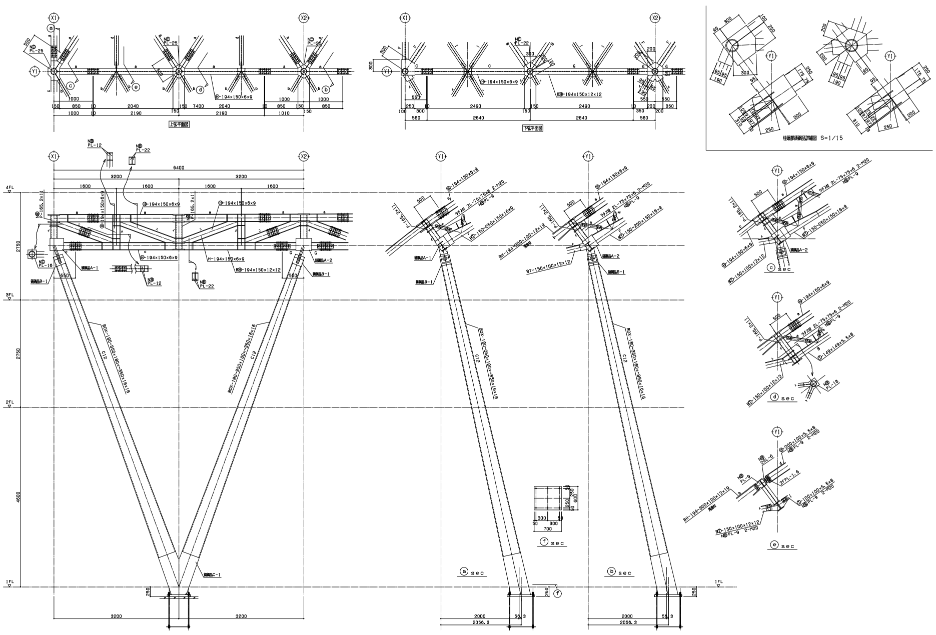
Photo 6 Wood-grid roof structure after jacking down

Huge Roof Installation Method Requiring High Accuracy

In the assembly of the huge roof, the truss beams were separated into their respective units, which were then subjected to field assembly. After confirmation of the assembly accuracy, these units were hoisted and assembled into specified positions. Further, after assembly of the huge roof, assembly accuracy was confirmed using surveying instruments.

In the installation of the huge roof, after first placing crawler cranes on both the mountain and reservoir sides, the steel frames were installed using these two cranes from the reservoir side toward the mountain side. Because V-shaped column head connections using clevis pins were adopted, strict accuracy was required to install the V-shaped columns, for which enhanced accuracy was secured. (See Photo 4)

Fig. 4 Detail of V-shaped Column



Outline of ROGIC Building

Location: Hamamatsu, Shizuoka Prefecture
Project owner: ROKI Co., Ltd.
Main application: R&D facility
Site area: 67,510.58 m^2
Building area: About 5,000 m^2
(incl. 1,500 m^2 of existing headquarters building)
Total floor area: About 9,000 m^2
(incl. 4,500 m^2 of existing headquarters building)
Structural type: RC structure
(partly steel frame-RC composite structure)
No. of stories: 4
Maximum height: 14,978 mm
Architect: Tetsuo Kobori Architects
Structural engineer: Arup
Mechanical and electrical engineer: Arup
Landscaping: studio on site
Lighting design: Izumi Okayasu Lighting Design
Office design: Okamura Corporation
Construction: Taisei Corporation
Design term: Jan. 2009~Oct. 2010
Construction term: Jan. 2011~Sept. 2013

Photos and figures

Photos 1, 3 and 7: Takahiro Arai
Photo 2: Kawazumi·Kenji Kobayashi Photo Office
Photos 4, 5 and 6; Figs. 1, 2, 3 and 4: Tetsuo Kobori Architects

After completion of the roof's steel-frame installation, the wooden members were attached, and then the completed roof was jacked down. The deformation level at each stage coincided well with the calculated values. (See Photos 5 and 6)

Certain approaches were incorporated in the design stage so that the columns to support the roof could not be seen in the work space, among which were the arrangement of the columns outside of the work space and their housing in the shutter rails and walls. The V-shaped columns to support the roof on the south side were also arranged outside of the work space.

Particular care was paid in the aesthetic design of the V-shaped columns. A cross section of these columns has a rhombus shape in which the box section is rotated by 45° to match the grid configuration of the roof. In elevation (Fig. 4), the column has a tapered shape (350×350 square in column base and 175×175 square in column head). Seam welds and on-site welds of the V-shaped columns were grinder-finished to impart these columns with a sculptural image.

In the connection between the roof and the columns, cast steel members were adopted to transfer the load acting on the roof to the columns. Also, particular care was paid to the configuration of the gusset plates inserted into the cast steel members so that the beautiful rhombus shape of the columns could be seen. (Refer to Fig.4 and Photo 7) ■

Photo 7
East-side look of the huge roof that spans an entire building

Seminar in Seven Cities for the Wider Application of Structural Steel

In order to promote the wider application of steel structures in building construction, the Japan Iron and Steel Federation (JISF) has yearly held a "Seminar on Steel Products for Building Construction and Their Application Technologies." From November to December 2016, JISF held a seminar in seven major cities in Japan: Tokyo, Osaka, Nagoya, Sapporo, Fukuoka, Hiroshima and Sendai. A total of about 450 persons from construction companies, design offices, fabricators, government agencies and universities attended.

In the respective seminar sites, lectures were delivered by experts in four areas—the JISF Committee on Building Construction, university professors of the seminar's host city, and researchers from the National Institute for Land and Infrastructure Management and the Building Research Institute.

Among the technical lectures delivered by the JISF Committee on Building Construction were "Guidebook for Preventing Brittle Fractures of Inner Diaphragm Electro-slag Welds" and the recently revised "Guidelines for the Design and Welding of SA440 (high-strength steel for building structures)." Their outlines are introduced in the current No. 52 issue of *Steel Construction Today & Tomorrow*.

JISF plans to hold the seminar in 2017 as well.

In September 2016, the Japanese Society of Steel Construction also held in Tokyo and Osaka a course that gives detailed explanations on the "Manual for Standard Tests for Mechanical Properties of Welds of Steel-frame Buildings" and other latest publications relating to design of steel-frame welds and execution of steel-frame welding. JISF and the Japan Steel Constructors Association participated in the course as co-sponsors.

JISF seminar in Tokyo

JISF seminar in Osaka

Presentation at SEAISI Conference under an Environmental Theme

The South East Asia Iron and Steel Institute (SEAISI) held the 2017 SEAISI Conference and Exhibition in May 2017 in Singapore. The JISF International Environmental Strategic Committee dispatched Committee Chairman Ken-ichiro Fujimoto and a member of the JISF staff to present two papers at the conference.

In the Cost & Energy Management II session of the conference, Committee Chairman Ken-ichiro Fujimoto presented a paper titled "Voluntary Energy Management in the Japanese Steel Industry," in which he discussed "Commitment to a Low Carbon Society," a program for voluntary energy savings and CO_2 emission reduction being promoted in the Japanese steel industry.

In the Environmental Management session of the conference, the JISF staff member delivered a paper titled "Life Cycle Assessment of Steel Products Incorporating Steel Recyclability," in which he introduced the development of international standardization of the LCI (Life Cycle Inventory) Calculation Methodology for Steel Products that the Japanese steel industry proposed in July 2015.

Presentation by Chairman Ken-ichiro Fujimoto of JISF International Environmental Strategic Committee

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The Japan Iron and Steel Federation
3-2-10, Nihonbashi Kayabacho, Chuo-ku, Tokyo 103-0025, Japan
Phone: 81-3-3669-4815 Fax: 81-3-3667-0245
URL http://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
URL http://www.jssc.or.jp/english/index.html

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Chairman (Editor): Hiroyuki Tanaka

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