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【 *c h o* 】

"超 (*cho*)" in Japanese, or "super" or "ultra" in English Extremely; Beyond a particular limit This issue, No. 38, features the advanced super high-rise building technologies in Japan.

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Japanese Society of Steel Construction

JSSC Prizes 2012 SkyPark: A Huge Rooftop, Steel Structure Spanning Three High-rise Towers

Prize winner: Yasuhisa Miwa, JFE Engineering Corporation

Outline of SkyPark

As its name implies, SkyPark is an aerial park constructed atop three high-rise towers in Singapore. It measures 340 m in total length and 40 m in width. As a new landmark, SkyPark celebrated its grand opening in June 2010 and now stands out among the various facilities operated by Marina Bay Sands Pte Ltd., an integrated resort company in Singapore (Photo 1).

A joint venture between JFE Engineering Corporation of Japan and Yongnam Engineering & Construction Pte Ltd. of Singapore was awarded the contract in April 2008 to construct the steelwork for SkyPark. The joint venture immediately commenced a detailed design and erection plan and in July 2009 started the on-site work to complete the erection of an 8,000-ton steel structure in a mere nine months.



Photo 1 SkyPark: A new landmark in Singapore

Structure of SkyPark

SkyPark is composed of two steel truss bridges connecting the three hotel towers (Towers 1, 2 and 3), a steel box girder bridge extending to form 67.7m-long cantilevered structure on Tower 3, and two steel-frame structures atop on Towers 2 and 3 to make an integrated structure.

Steelwork Fabrication and Erection

Steel structural members were fabricated by Yongnam Engineering & Construction, a local fabricator, and transported to the construction site.

In erecting the steel-frame structures on Towers 1 and 2, the structural members were lifted one by one using the tower crane to place them at the specified positions. (Refer to Photos 2 and 3)

Structural members for the box girder bridge on Tower 3, the two tower-connecting

bridges and the cantilevered structure on Tower 3 were pre-assembled into large segments at the ground level in the side of the tower and lifted by means of a heavy-duty strand jack assembled on a gantry frame installed atop the tower. The large segments thus assembled numbered 3 each for the main girders of the two tower-connecting bridges, 2 for the two main girders of the box girder on Tower 3, and 6 for cantilevered structure. A total of 14 large segments, weighing 4,000 tons in total, were lifted and erected during the three months from October 1, 2009 to December 29, 2009. The segments were lifted to the target height of 200 m. Each segment was lifted at a rate of 15 m/hour, over a 15-hour period.



Photo 2 Heavy lifting of steel truss bridge segment



Photo 3 Heavy steel segments were lifted to the target height of 200 m.

Safety Measures

Because the construction work for SkyPark routinely took place at heights surpassing 200 m as well as ground with other trades, unprecedented care was taken to secure maximum worker safety. Practically speaking, utmost efforts were thoroughly and repeatedly made to spread over a large working group, including 450 workers and 70 staff during the busiest period, that all workers and staff wear a full harness to prevent falling and carry lanyards to prevent the dropping of tools. (Refer to Photo 4)

* * *

A noteworthy accomplishment in the construction of SkyPark is the completion of such a huge project, involving difficult construction work, without serious accidents over one million of working hours of labor. This is largely attributable to the united and highly vigorous effort by local enterprise staff, locally recruited engineers and the Japanese staff, efforts that transcended culture and language, to successfully complete the project.



Photo 4 A huge project was completed without serious accidents over one million of working hours of labor.

Amagasaki Research and Development Center—Employing 1,000 N-grade Steel

Prize winners: Nikken Sekkei Ltd., Shimizu Corporation, Katayama Stratech Corporation and Sumitomo Metal Industries, Ltd. (now Nippon Steel & Sumitomo Metal Corporation)

Sumitomo Metal Industries, Ltd. (now Nippon Steel & Sumitomo Metal Corporation) has developed 1,000 N-grade high-strength steel (tensile strength: 950 N/mm²), which is rated as the world's highest grade of steel. The first practical application of this steel was in the construction of the main building of the company's Comprehensive Research and Development Center (currently the Amagasaki Research and Development Center of Nippon Steel & Sumitomo Metal). (See Photo 1)

1,000 N-grade steel was developed by a joint R&D consortium composed of Sumitomo Metal, Osaka University, the Kyoto Institute of Technology, Katayama Stratech Corporation and Nikken Sekkei Ltd. The cooperative R&D was conducted from 2003 to 2010 and covered not only development of the steel itself and related welding materials but also welding conditions, such as heat input, preheating and post-heating.



Photo 1 Appearance of the building

The building offers the following three major structural features:

• The response-control structures are concentrated in the first story and are composed of 1,000 N-grade steel columns and response-control braces (Fig. 1). Most of seismic energy of an earthquake is absorbed by the response-control braces



Photo 2 Steel framing installation



Photo 3 Column base



1,000 N-grade steel column

Response-control brace





Photo 4 Interior view of laboratory room

installed in the first story, and thus the frames of the second and higher stories remain in an elastic state during large-scale earthquakes. In order that 1,000 N-grade steel columns remain in an elastic state during seismic motion that surpasses assumed levels, the base of each column is fitted with spherical supports and short brackets to provide a trigger mechanism that prevents the columns from yielding (Fig. 2, Photos 2 and 3).

- A wide working space, 133 m × 23 m, is provided so that the laboratory can flexibly adjust to future changes in research themes and research organization (Photo 4).
- A double-skin structure is adopted on the east side to promote energy savings, and air-conditioning and

lighting systems have been adopted that take into a c c o unt the o p e r a t i n g environment of the laboratory rooms.

When the building is viewed from the outside, the steel column-beam framing that is assembled in a lattice shape is visible through the glass, and at the first-story entrance, visitors can directly view the 1,000 N-grade steel columns and the response-control braces concentrated on the first story. These two structural systems account for the notable features of the building's appearance.

Technological Development of Countermeasures against Long-period, Long-duration Seismic Motion in Existing High-rise Buildings

Prize winners: Osamu Hosozawa, Yuichi Kimura, Kenji Suda, Chiaki Yoshimura and Hideshi Aono, Taisei Corporation

It is forecasted that Tokai, Tonankai, Nankai and other great earthquakes will occur in the near future in Japan. In this regard, there is concern that long-period, long-duration seismic motions occurring during these earthquakes will be transmitted from the epicenter to far reaching areas while retaining a great amount of energy. In large urban areas, there are many high-rise buildings and some of these existing buildings were designed without consideration of countermeasure against long-period seismic motions. Should these buildings be subjected to long-period seismic motions, there is apprehension that large oscillations will continue for a long time and that these oscillations will damage not only building structures but also non-structural members and equipment.

As a measure to treat long-period seismic motions occurring in existing high-rise buildings, it is effective to install response-control dampers so as to add damping capacity and to thereby reduce maximum deformation and lateral oscillation. However, the installation of dampers on existing buildings poses one problem: the reaction force of the dampers acts on the existing framing (columns, beams, foundations etc) which, as a result, requires reinforcement.

The countermeasure technology that we have developed for long-period, longduration seismic motions in existing highrise buildings adequately meets this problem. Specifically, in applying this technology, deformation-dependent oil dampers are adopted to reduce damping capacity in the vicinity of maximum deformation, thereby allowing the installation of dampers without reinforcing the existing framing. Further, a crimping method has been developed for attachment of the dampers that uses PC steel bars and does not require on-site welding, which has facilitated damper installation work on target buildings while in use. (Refer to Fig. 1 and Photo 1)

This technology was applied to the Shinjuku Center Building as a countermeasure against long-period seismic motions (Photo 2). As a result, during the Great East Japan Earthquake of March 2011, this building did not suffer any notable seismic damage thanks to the response-reduction effect brought about by the use of deformation-dependent oil dampers and was able to continue business operations without interruption.

Fig. 1 Installation Position for Oil Dampers



Photo 2 Shinjuku Center Building

Framing elevation



Tokyo Gate Bridge: Truss-Box Composite Structure

Prize winner: Ken Fukunishi, Tokyo Port Office, Kanto Regional Development Bureau, Ministry of Land, Infrastructure, Transport and Tourism

The Tokyo Port coastal highway project was planned for three reasons: to alleviate traffic congestion in the area surrounding the Port of Tokyo that resulted from increased cargo container activity, to secure smooth logistics and to reduce logistical costs. The Tokyo Gate Bridge is a 2,618 m-long steel bridge that constitutes one segment of the highway (Photo 1).

Being located near Tokyo International Airport (Haneda), airspace regulations of 98.1 m is settled. Further, because the bridge spans the east fairway of the Port of Tokyo, it had to provide unobstructed passage space of 300 m in width and 54.6 m in height above water level. Accordingly, a truss bridge was selected because a suspension bridge or a cable-stayed bridge requiring main towers would be untenable.



Photo 1 Truss-box composite structure Tokyo Gate Bridge

Design of the Tokyo Gate Bridge

Construction of the bridge rested on design work that had to hurdle a variety of restricting conditions, reduce construction costs, create good landscaping and suppress maintenance costs. To meet these requirements, it was necessary to develop and adopt new technology, collect bridge d a m a g e e x a m p l e s a n d d e v i s e damage-improvement measures. Some representative examples of these efforts follow:

• Reduction of Steel Weight by Use of BHS Members: For truss bridges in which the dead load accounts for most of a bridge's weight, it is accepted that construction costs can be lowered by reducing the bridge weight to a minimum. Because BHS (bridge high-performance steel) products have greater strength than conventional steel products, their use allows not only for a



Photo 2 En-bloc installation of the 6,800-ton substructure assembled on-land

reduction in the total weight of the steel used but also for the efficient construction of bridges built under restrictive natural conditions and requiring narrower welds in the construction. Because of these, BHS products were extensively used.

• Rationalized Design of the Steel Slab Framing and Fatigue Counter-

measures for the Steel Slabs: Having conducted studies of truss frame structures that could reduce superstructure deadweight and rationally distribute live loads, a composite truss frame of the box girder type was selected. In order to prevent the concentration of stress caused by cyclic loading on the steel slab, the trough rib configuration was improved to produce a steel slab with high fatigue strength.

• Rationalized Panel Point and Fullsection Welding: The adoption of rigid joining by means of full-section welding of truss panel point, not by pin joining, eliminates the need for splice plates and bolts, which, in turn, produces a structure that is corrosion resistant and cheaper in maintenance cost.

• Bearings with Seismic-isolation Structure: In order to support the 80,000 kN of bearing's vertical reaction force and to provide seismic resistance during an earthquake, function-separated, sliding seismic-isolation bearings were adopted. The load support function and the seismic resistance function of the bearings were separated, and two devices were installed: a seismic-isolation rubber bearing that absorbs the horizontal load occurring during an earthquake, and a load supporting plate that constantly bears the vertical load and slides in conformity with the behavior of the seismic-isolation rubber bearing during an earthquake.

• Bridge Monitoring System: Because full attention has been given to bridge maintenance since the start of construction, a monitoring system was introduced that observes the consistency between the design and the bridge's current state, registers the characteristic properties of the live load and measures the bridge's accumulated damage. Specifically, strain gauges, displacement gauges and accelerographs were applied to quantitatively observe the constant numerical values and to obtain data conducive to working out future maintenance plans.

Construction of Tokyo Gate Bridge

Construction of the foundation structure began in October 2003. While the foundation and substructure work was carried out at the offshore site, manufacture of the superstructure was conducted at a land-based plant. In September 2008, the lower trusses (6,800 tons) that had been assembled on-land were installed en-bloc. The trusses were lifted in tandem onto the bridge piers by means of three 4,000-ton crane ships—the largest scale operation of its kind ever seen in Japan (Photo 2).

The superstructure was then installed span by span, until February 2011 when the center span, the last piece of the operation, was installed. After fence work and paving, the bridge was put into service on February 12, 2012. (Refer to Photo 3)



Photo 3 Illuminated Tokyo Gate Bridge, a wonderful nighttime view in the Port of Tokyo

JSSC Thesis Prizes 2012

Study of the Fracture Behavior of Electro-slag Welds in Beam-Column Connection Framing Models

Prize winners:YongHun Song, Kawagishi Bridge Works Co., Ltd.; Takumi Ishii and Hiroumi Shimokawa, JFE Steel Corporation; Takahiko Suzuki and Yoichi Kayamori, Nippon Steel & Sumitomo Metal Corporation; Yukihiro Harada, Chiba University; and Koji Morita, Tokyo Denki University



YongHun Song 2006: Entered Kawagishi

Bridge Works Co., Ltd. 2011: Finished the Graduate School & Faculty of Engineering,

Chiba University: Obtained D.Eng.

Along with the increasing height of building structures, there is a growing demand for weld built-up box-section columns. In the manufacture of these columns, electro-slag welding (ESW) is used for the welded joints between the column skin plates and the interior diaphragms. While ESW is a highly efficient welding method, it requires a large heat input. Accordingly, fracture toughness in the heat-affected zone of ESW welds is lower than that in the base metal, thereby causing concern that brittle fracture might occur (Fig. 1). Furthermore, because the adopted weld detail generates slits between the column skin plates and backing plates, the stress concentration is severely high at tips of the slits (Fig. 2), and thus it is difficult for the weld detail to adequately prevent the occurrence of brittle fracture.

In this paper, in order to clarify the fracture toughness of the weld bond, the heat-affected zone and the weld metal of ESW welds, several factors were considered with regard to the welding quality (penetration depth, misalignment) and design (tensile axial force of column skin plate). How these factors affect brittle fracture was introduced by means of experiments and analysis. In addition, state-of-the-art investigations were conducted to correlate the fracture toughness (resistance) with the fracture toughness index (requirement) based on the experimental and analytical results. More specifically, focusing on the critical maximum principal stress, the following two correlations, the critical maximum principal stress obtained by finite element analysis versus fracture toughness (Fig. 3), and the connection strength of the ESW joints obtained by experiments versus fracture toughness (Fig. 4), were found.



Plastic Deformation Capacity of Cold-formed Circular Steel Tube Columns Subjected to Cyclic Loading

Prize winner: Nobuyuki Yasui, Professor, Nagasaki Institute of Applied Science



Nobuyuki Yasui 1993: Finished

School/Graduate School of Engineering, Osaka University 1993: Entered Kawatetsu

Metal Products & Engineering Inc. 2011: Professor, Nagasaki Institute of Applied Science

The main object of this study is to clarify the plastic deformation capacity of circular steel tube columns, the strength of which is steadily reduced due to the accumulation of local buckling deformation caused by cyclic loading. For that purpose, numerical analysis was conducted on cold-formed circular steel tube columns subjected to cyclic loading with specified amplitudes under specified axial force conditions.

The analysis focused on circular steel tube columns subjected to cyclic horizontal deformation with specified amplitudes under specified axial forces P (Fig. 1), for which an asymmetrical local buckling analytical model proposed by the author was adopted. The relation between the member end moment M and the member drift angle θ shown in Fig. 2 was obtained from the analytical results, and $\Sigma \theta_{PM}$, the accumulated plastic drift angle up to the point where M fell to the full plastic moment M_{pc} that takes into account the axial force, was set as the plastic deformation capacity in the current study.

Fig. 3 shows an example of the relation between $\Sigma \theta_{pM}$, the accumulated plastic drift angle, and D/t, the width to plate thickness ratio. Because the relation between D/t and $\Sigma \theta_{pM}$ showed a favorable correlation, the relational expression of D/t and $\Sigma \theta_{pM}$ that assesses the plastic deformation capacity was found by regressing the analytical results.

The red circle in Fig. 3 shows the results of the experiment that was conducted to confirm the validity of the regression curve. Because the plotted points of the experimental results are distributed in the vicinity of the regression curve, it can be said that the plastic deformation



between D/t and $\Sigma \theta_{pM}$ as proposed in the current study.



Effect of Weld Shape on the Occurrence of Ductile Cracks in Welds Subjected to Bending during Earthquakes, and the Suppression of Ductile Cracks by Means of Weld Toe Finishing

Prize winners: Hiroshi Tamura, Assistant Prof., Tohoku University; Eiichi Sasaki, Associate Prof., Tokyo Institute of Technology; Songdong Kim; Hitoshi Yamada, Prof., Yokohama National University; and Hiroshi Katsuchi, Prof., Yokohama National University



In steel structures built using many welds, remarkable stress triaxiality and strain concentration occur due to weld shape discontinuity, thereby allowing the possible initiation of ductile cracking in weld toes because of bending deformation in welds during earthquakes. In such cases, there is the danger that ductile fractures will serve as the starting point of brittle fractures (brittle fractures during earthquakes). This type of brittle fracture occurred in many steel buildings during the Northridge and Great Hanshin Earthquakes, causing societal concern about steel-structure construction.

In the current study, in order to verify the effect of weld shape on limiting ductile crack initiation in weld toes, cyclic loading tests and FEM analysis were conducted on cruciform weld joint test specimens prepared by simulating the welds of actual steel structures. As a result, differences were experimentally shown regarding the limiting of ductile crack initiation under cyclic loading conditions depending on various weld shapes. Further, as a result of FEM analysis of the effect of weld shape confirmed in the tests, it became clear that, among leg ratio, toe radius and toe finishing and their respective effect on the reduction of base metal thickness, the effect of toe radius was particularly notable. From the above results, it is considered that the use of toe finishing to increase toe radius is effective in suppressing the development of ductile cracks occurring in weld toes.

Fig. 1 Test Specimen



Fig. 2 Experimentally-obtained Crack Initiation and Development (Values in the figure indicate the weld toe radius in mm)







Special Feature: Advanced Super High-rise Building Technologies in Japan A Seismic Design of High-rise Buildings

by Akira Wada, Professor Emeritus, Tokyo Institute of Technology; Chairman of CTBUH Japanese Group

Development of Seismic Engineering in Japan

In 1924, one year after the Great Kanto Earthquake that devastated Tokyo, Professor Toshikata Sano (1880-1956) added to the Urban Building Law a new requirement: the static horizontal seismic factor should be set as 0.1 or more. Ten years later, Professor Ryo Tanabashi (1907-1974) published an article in July 1934 stating that the seismic resistance of a structure cannot be adequately assessed simply by providing ample strength against a static horizontal force; he contended that the seismic impact should be expressed using the energy squared by the maximum ground velocity and that the resisting capacity of a structure should be assessed using the strain energy absorbed by the structure itself. In March of the same year, he suggested that research should be started on the construction, even in earthquake-prone Japan, of super high-rise buildings like those seen in New York.

In the postwar period, Professor Tanabashi insisted through his article published in April 1963 that high-rise building should be possible in Japan in light of the following examples: given that seismic motion works on small and large structures with identical amplitudes, a flower vase might fall over in an earthquake, but a large high-rise building would not, even if both objects were proportionally identical. In other words, contrary to small boats, large ships are resistant to capsizing in rough seas.

Around the same time, Professor Kiyoshi Muto (1903-1989) promoted research on a high-rise building for the Tokyo Station. While his effort in this case was not rewarded, the Hotel New Otani was completed in Tokyo in 1964 with a building height surpassing 45 meters. In 1968, the Mitsui-Kasumigaseki Building designed by Prof. Muto, was completed as Japan's first high-rise building to surpass 100 meters in height. On every story of the building frame, precast concrete walls with many vertical slits were incorporated to maintain their initial structural stiffness while absorbing energy during a strong earthquake. Accordingly, it can be said that the concept of passive-controlled structures was already being applied at the initial stage of high-rise building in Japan.

Introduction of Advanced Seismic Design Entering the 1970s, most high-rise buildings were constructed using a seismic design method that relied on the plastic rotation capacity of steel-frame beam ends to provide energy absorption. However, several structural designers believed such designs would leave these buildings with residual deformation in frames subjected to large plastic deformation, thereby making restoration difficult. In response to this, the concept of damage-controlled design began to grow (refer to the figure below). The Northridge Earthquake of January 1994 and the Great Hanshin Earthquake of January 1995 caused fracture phenomena in many steel-structure beam-ends, resulting in considerable concern about the feasibility of restoring damaged buildings.

In Japan, following implementation of the New Seismic Design Codes in June 1981, extensive research has been conducted on seismic-isolation structures. In 1995 a seismic-isolation structure was put into practical use that adopted energy-absorption members such as steel and lead dampers along with employing laminated rubber bearings as elastic supporting members. Since then, another concept has been increasingly applied whereby the beam-column frames of high-rise buildings bear vertical loads in a manner similar to the laminated rubber bearings in seismic-isolation structures. This design produces mainly elastic behavior during an earthquake so that the seismic energy is absorbed by the energy absorbing members incorporated in the framing of each floor.

Enhanced Seismic Resistance

In addition to the seismic designs mentioned above, the seismic safety of high-rise steel structures is steadily being enhanced due to

following factors: higher strength and sufficient ductility of steel materials, the provision of upper and lower limits for yield stresses, progress in welding technology and the adoption of haunches to prevent the plasticization of beam-end welds. Another contributing factor is the utilization of column members with stiffness and strength that are made possible by the use of concrete-filled tubular columns manufactured by filling square or circular steel tubes with concrete.

In addition, remarkable progress in computer-aided structural analysis technology makes it possible to use dynamic response analysis that can accurately treat the dynamic behavior of columns, beams, shear walls and various dampers. This, in turn, has resulted in the construction of high-rise buildings with complex framing and super high-rise buildings with heights reaching 300 meters.

The Great East Japan Earthquake that occurred on March 31, 2011, produced large amplitudes and long-duration vibrations in high-rise buildings not only in Sendai near the epicenter but also in Tokyo, Osaka and Nagoya. Earthquake ground motion is recorded by means of acceleration, and response analysis for seismic design is conducted by inputting the recorded ground motion. However, because the acceleration of the long-period component included in ground motion is much lower than that of the short-period component, certain problems have emerged vis-à-vis the occurrence of unexpected large vibrations in buildings located far from an epicenter. These large vibrations are due to the following two reasons: 1) the earthquake ground motion is not accurately included in the acceleration records applied in conventional design; and 2) Earthquake ground motion is commonly applied for 30 seconds duration in the response calculation. However, tall structures with limited damping capacity that are subjected to long-term ground motion with a period of 4 seconds or more exhibit synchronized large vibrations after this 30 seconds until 600 seconds. To cope with this problem, existing high-rise buildings have been retrofitted with seismic energy absorption members.

Damage-controlled Structure



Wind-resistant Design of High-rise Buildings

by Yukio Tamura, Professor, Tokyo Polytechnic University; Member of CTBUH Japanese Group

Points to Note in Wind-resistant Design of High-rise Buildings in Japan

In Japan, very strong seismic excitations such as The 2011 Off The Pacific Coast Tohoku earthquake on March 11, 2011 (magnitude 9.0, maximum recorded ground acceleration 2,933 cm/s²) have to be considered. It is also true that Japan has very strong typhoons, e.g. Typhoon Maemi passed over the Miyakojima Island on September 10 and 11, 2003, and a 3s gust wind speed exceeding 90 m/s was recorded (Cao et al., 2009).

For seismic actions, buildings should be lightweight and flexible, but for wind actions, buildings should be massive and rigid. Thus, opposite design criteria apply for buildings for seismic and wind actions, and very high levels of both seismic actions and wind actions have to be considered in Japan.

In general, the dominant external design load is seismic load for the majority of high-rise buildings, say lower that 200 m high. Therefore, they are basically lightweight and flexible, thus making them vulnerable to winds. Furthermore, habitability to building vibrations induced by daily winds is inevitably an important issue in Japan.

Since the early '70s, unique and significant development has been made in Japan not only for structural performance against external actions, but also for evaluation of habitability to building vibrations and for wind environmental assessment. For various wind resistant design aspects, suppression of wind-induced responses is an important issue.

Comprehensive Study on Aerodynamic Characteristics of High-rise Buildings with Unconventional Configurations

For high-rise or super-high-rise buildings, crosswind response due to periodic Karman vortex shedding is predominant over along-wind or torsional responses. Therefore, aerodynamic means to prevent formation of Karman vortices, to reduce their intensity and periodicity, and to minimize spatial correlation of shed vortices along the vertical axis, are useful. Recently, many high-rise and super-high-rise buildings with unconventional configurations, such as Burj Khaliha and Shanghai Tower, have been constructed around the world. One reason for their curious and complicated configurations is the advantageous aerodynamic characteristics, especially for the crosswind component.

The author has been collaborating with Takenaka Corporation, Kajima Corporation, Nikken Sekkei, and Nihon Sekkei, to comprehensively investigate aerodynamic characteristics of super-high-rise building models with various unconventional configurations. This work has been financially supported by the Ministry of Land, Infrastructure, Transport and Tourism (2008FY-2009FY, MLIT) and Japan Science and Technology Agency (2010FY-2012FY, JST). Thirty-seven models including square models with sharp edges, with corner cut, and with corner chamfer; models with various twist angles (helical models), with setbacks, with taper, and with various openings; and triangular models including clover type, composite types, and so on. The height and the volume were set at 400 m and 10⁶ m², respectively, for all models.

According to our study, the aerodynamic characteristics of triangular models and a clover type model, which have recently become popular for super-high-rise buildings, are not necessarily good. Fig. 1 shows the maximum along-wind and crosswind fluctuating wind force coefficients for 28 models, except for triangular and some other models. "Square Model" is shown on the left end of Fig.1. The efficiency of corner modifications, twisting angle for helical models, taper, setback, and combinations of these configurations are clearly demonstrated. Combination effects are especially significant for reducing crosswind forces.





Latest High-rise Project

Sony City Osaki —Transpiration Louvers and Variable Seismic-isolated Structures—

by Toshihiko Kohno, Nikken Sekkei Ltd.

In order to treat the emerging problem posed by "urban heat islands," the Sony City Osaki project was based on the idea of "constructing a building as if planting a tree." In addition, the building plan was started with an awareness of the following question: What kind of advanced environmental measures can be used by such a large-scale building project that generates a large environmental burden?

Full Consideration of Environmental Issues

In order to allow the wind to blow through, the building has the shape of a flat vertical plane (Photo 1) and is landscaped with rich greenery in the surrounding area. It also makes trial use of a newly developed "bio skin" system installed on the east side of the building. In the system, rainwater is circulated inside transpiration terra-cotta louvers that allow the rainwater to evaporate, which takes heat from the air. In addition, the wind blows through the building lot, thereby cooling the surrounding area of the building (Photo 2). The power required to circulate the rainwater is provided by solar panels.

Actual measurements confirm that



Photo 1 Full view of Sony City Osaki

ambient peripheral temperatures were reduced almost as forecasted by simulations conducted in the design stage using building information modeling (BIM). The bio skin is supported by a tension structure using stainless steel rods. The specific tension introduced by means of nut rotations is confirmed through vibration measurement.



Photo 2 "Bio skin" system in which rainwater is circulated inside transpiration terra-cotta louvers to cool the surrounding area of the building

Outline of Building Plan

The flat vertical plane shape of the building was an inevitable choice from an environmental perspective. Thus, the building is designed as a single core structure in which the 24 m \times 130 m office floors are located on the east side and the EV, stairs, restrooms and other equipment are placed on the west side. The workspaces have an area of about 3,000 m² each and are column-free due to the use of external framing. Balconies around the building periphery help to improve maintenance performance and fire evacuation safety. (Refer to Fig. 1 and Photo 3)

The facilities for various functions along the west-side core are positioned as needed, and five void cores have been left empty to meet the need for future changes in function. Space is provided below ground for a 3,400-ton heat storage tank and for sodium–sulfur (NAS) batteries to be used in emergencies and for energy savings.







Photo 3 Spacious column-free workplace

Seismic-isolation System

A major structural theme of the current project was to secure improved seismic resistance by adopting seismic-isolation devices appropriate for an office building in the shape of a thin vertical plane having a height over 130 m. (Photo 4) The seismic resistance of the building during great earthquakes is twice that of non-seismic isolation-type buildings (an approximate inter drift angle of 1/200 and a floor response acceleration of 250 cm/s² or lower). The adopted variable seismic-isolation system is composed of natural rubber-laminated isolators (high bearing specifications of 1,100~1,500 ϕ mm), oil dampers and steel dampers.

Because of the steel-structure flat vertical shape of the building, the anticipated wind load generated by a typhoon would be larger than the seismic load reduced by the seismic-isolation system. The oil dampers installed on the narrow side of the building that is subject to large wind loads are fitted with a locking mechanism to control oil movement using electromagnetic valves (Photo 5). The oil dampers themselves are controlled using anemometers, accelerometers and displacement gauges to mitigate the vibration of the seismic-isolation layer and the accumulated plasticity of the steel dampers during strong winds. Seismic motion-precedence control is adopted so that the locking system is released when a certain level of seismic motion is detected. The combined use of oil and steel dampers is effective in securing structural rigidity when oil movement is locked and is a solution that resulted from the pursuit of low cost and high performance.

The beams on the west-side core face

cantilever more than 7 m and reduce the axial force of the intermediate columns so as not to overstress the 1,500 ϕ mm isolators supporting the intermediate column.

The top of the cantilevered beams that are greater than 7 m in length are connected by studs to a viscous material so as to reduce vertical vibration and relative displacement. In addition, other vibration countermeasures have been taken at various levels: improvement of damping capacity by installing V-shaped visco-elastic members on the short vertical face (Photo 6) and micro-vibration countermeasures on the fifth floor.

The Sony City Osaki project employs the most advanced approaches for high-rise building construction that have ever been



Photo 6 V-shaped visco-elastic damper to control vibrations

proposed and put into practical use—including a seismic-isolation system of variable mechanisms and a new environmental method for cooling a building's immediate vicinity. (Refer to Photos 7~9)

> All photos of Sony City Osaki: Harunori Noda (Gankosha)



Photos 7~9 Sony City Osaki project was based on the idea of "constructing a building as if planting a tree" in order to treat the emerging "urban heat island" problem.



Photo 4 Construction of the seismic-isolation floor



Photo 5 Oil damper fitted with a locking mechanism to control oil movement

ABENO HARUKAS -Performance-based Design of Japan's Tallest Building—

by Kiyoaki Hirakawa, Takenaka Corporation

This article introduces specific examples of how performance-based design was incorporated in the overall structural design of ABENO HARUKAS, the tallest building (300 m) in Japan, consisting of a department store, offices and a hotel (Photo 1).

Outline of Structural Plan

Five major issues had to be solved in the structural design of the building.

- Constructing a frame capable of handling high axial forces
- Designing a framing plan to suppress deformation
- Responsiveness to foreseeable largescale seismic shocks
- Securing acceptable comfort levels in strong winds
- Introduction of technology that considers construction efficiency

The structural plan that was devised to resolve these issues are outlined below:

The truss floors are located between the low-rise and medium-rise sections and between the medium-rise and high-rise sections; and, further, braces are positioned in the mid-story office spaces so as to obtain a reverse bending effect (Fig. 1).

In the low-rise section, viscous oil dampers and rotational friction dampers are arrayed in the four corners. In the vicinity of the center core of the mediumrise section, corrugated steel plate seismic walls are placed along the X direction and seismic steel-frame braces in the Y direction. In the high-rise section, a rigid frame is adopted in the X direction; and in the Y direction, seismic steel-frame braces are located inside the boundary walls of the hotel rooms.

Diverse Types of Seismic Design

The building's seismic design incorporated three categories of seismic shock input, as shown in Table 1. The first category displays the values for legally required waves and conventionally adopted standard waves; the values of the second category, representing the maximum levels of seismic shock (used for seismic safety margin analysis level), are obtained by multiplying Level 2 seismic



Photo 1 Latest construction situation (north-west view)

shock waves by 1.5; and the third category, taking localized characteristics into consideration, displays the seismic shock waves asso-



ciated with the Tonankai and Nankai Earthquakes and the Uemachi Fault.

As shown by the seismic design criteria in

Table 1 Input Seismic Motions for Design Purpose and Seismic Safety Margin Analysis							
		Level 1		Level 2		Seismic safety margin analysis level	
		V max. (mm/s)	A max. (mm/s ²)	V max. (mm/s)	A max. (mm/s ²)	V max. (mm/s)	A max. (mm/s ²)
	Elcentro 1940NS	250	2,555	500	5,110	-	-
Standard wave	TAFT1952EW	250	2,485	500	4,970	-	-
	Hachinohe 1968NS	250	1,669	500	3,338	-	-
	Notification wave A	76	552	379	2,517	528	3,244
Notification wave	Notification wave B	109	471	613	1,929	817	2,919
	Notification wave C	83	497	383	2,098	560	2,604
	Nankai Earthquake NS	-	-	210	863	-	-
	Nankai Earthquake EW	-	-	275	987	-	-
	Tonankai/Nankai Earthquakes NS		-	-	-	268	1,213
Regional	Tonankai/Nankai Earthquakes EW	-	-	-	-	419	1,067
wave	Uemachi fault zone Case 1 NS	-	-	-	-	329	3,485
	Uemachi fault zone Case 1 EW	-	-	-	-	813	4,107
	Uemachi fault zone Case 2 NS	-	-	-	-	291	2,317
	Uemachi fault zone Case 2 EW	-	-	-	-	456	3,269

Fig. 1 List of Technologies Adopted

Table 2 Design Criteria						
Seismic motion		Level 1	Level 2	Seismic safety margin analysis level		
		Rare About 50 years of recurrence interval	 Extremely rare About 500 years of recurrence interval 	1.5 times stronger than notification level 2		
		Continuously usable	Repairable	Repairable (Reinforceable)		
	Story drift angle	5.00×10 ⁻³ rad.	10.00×10 ⁻³ rad	13.50×10 ³ rad		
Super- structure	Ductility factor of story	Allowable stress for short-term loading, or less	1.0 or lower	2.0 or lower		
Sub- structure	Underground frame and foundation	Allowable stress for short-term loading, or less	Less than ultimate strength	Underground frame: Ductility factor of members<4.0 Foundation/piles: Ultimate strength or lower		
	Pile bearing capacity	Allowable bearing capacity for short- term loading, or less	Allowable bearing capacity for short- term loading, or less	Ultimate bearing capacity or less		

Table 2, the criteria adopted for the current building have been upgraded from the design criteria used for structural members used in ordinary high-rise buildings. Specifically, these members do not allow plasticization in response to Level 2 seismic movement, but in response to earthquakes in the range where degrees of allowance are studied, plasticization of the girders and braces is allowed.

Application of New Structural Technologies

• CFT Columns Employing High-strength Materials

In order to bear high axial forces, concrete-filled steel tube (CFT) columns were adopted that use both high-strength Fc150 concrete and 590 N/mm²-grade high-strength steel. CFT column performance was thoroughly confirmed by means of not only structural performance confirmation tests but also fire resistance tests and concrete filling performance tests (Photo 2).



Photo 2 Filling capacity confirmation test

New Joining System

A division-type outer diaphragm system (Fig. 2)¹⁾ was adopted in consideration of the concrete filling performance of CFT columns and the work efficiency of the CFT column manufacturing plant. In the joining of girders to the CFT columns, a new connection system was adopted that uses

sprayed aluminum to improve the sliding coefficient from 0.45 to 0.70^{2} , thereby reducing the labor required for joining and reducing the construction term.

Fig. 2 Division-type diaphragm



Response-control Dampers

In the low-rise section of the building, both highly velocity-dependent oil dampers and highly deformation-dependent rotational friction dampers were installed.

The rotational friction dampers (Photo 3) have the advantage of being a simple mechanism and using friction as the damping force, thereby allowing the prescribed friction force to be obtained by adjusting the piling number of the pad without taking brace rigidity into account.

In the medium-rise section, corrugated steel plate walls are used in the walls of the EV halls and resist the horizontal forces by allowing shear yielding. They effectively absorb seismic energy (Photo 4).



Photo 3 Front view of rotational friction damper



Photo 4 Corrugated steel plate wall

Fig. 3 Guide Damper







In the high-rise section, a vertical truss drops downward from the hat truss toward the atrium (Fig. 3) and oil dampers are installed between the downward truss and the peripheral framework, thereby reducing the inter-story drift angle of the high-rise section by about 10%. An active tuned mass damper (ATMD) is installed at the top of the building (Fig. 4).

The hat truss floor is equipped with mass dampers, one each at the west and east ends of the building as shown in Fig. 4 for the purpose of improving the habitability of the hotel in strong winds³⁾.

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ARK Hills Sengokuyama Mori Tower —Advanced Safety and Building Technologies—

by Toru Tsuchihashi and Masaharu Yasuda, Mori Building Co., Ltd.; and Masayuki Yamanaka, Shokichi Gokan and Shuichi Otaka, Obayashi Corporation

The ARK Hills Sengokuyama Mori Tower represents one element in the large-scale Toranomon-Roppongi District Urban Development Project being promoted in the Toranomon and Roppoingi areas of Tokyo. The Tower development project covers about 2.0 ha and was completed in August 2012. Surrounding the redevelopment area are many embassies and hotels that anchor the area's role as an important international and cultural center in Tokyo.

The 47-story complex sits on relatively high ground and consists of commercial

Photo: Toranomon-Roppongi Work Office



Photo 1 Panoramic view of ARK Hills Sengokuyama Mori Tower

Table 1 Building Outline

Use	Office, residence, retail, parking
Site area	15,367.75 m ²
Total floor area	143,550.04 m ²
Floors	47 above ground, 4 under ground
Maximum height	+206.69 m
Structure	Reinforced concrete (partly: steel) Seismic control structure

and residential sections (floors 1-24) and an office section (floors 25-47). A separate 7-story residential building (seismicisolated structure) is planned for erection on the south side of the complex. (Refer to Photo 1 and Table 1)

Outline of Structural Type and Design

Rigid reinforced concrete framing was adopted for the building's structure. The standard floor plan was 50.4 m \times 50.4 m (7.2 m \times 7 spans). For the high-rise office floors, steel girder framing with a length of 2~3 spans was adopted to realize wide,

Fig. 1 Standard Floor Framing Plan for Office Floors



(Source : Obayashi Corporation)

column-free spaces (Fig. 1). In the center core section, viscous response-control walls and hysteresis response-control walls (brake dampers) were installed (Photo 2).

The main structural materials were high-strength concrete (Fc120 N/mm² max.) and high-strength steel reinforcing bars (σ y685,785 N/mm²). The foundation type is of the spread foundation consisting of 4.5 m-thick mat slabs, and was installed on a supporting layer of sandy soil (longperiod soil bearing strength: 100 t/m² or more). The structural performance targets of the building and the corresponding test results are shown in Table 2.



Photo 2 Viscous response-control wall and hysteresis response-control wall (brake damper)

Precast Method LRV Method

In the LRV (left right vertical installation precast) method, connection-partial beam precast members are adopted. The precast members are of two types: beam-connection

Table 2 Structural Performance Targets and Corresponding Test Results

		Target volue	Results				
Level		Talget value	Direction	Calculated value	Generating floor	Seismic wave	
	Drift angle	R<1/200	0°	1/349	28F	Taft-EW	
Level 1			90°	1/413	26F	Taft-EW	
	Ductility ratio	µ<1	Yielding does not occur				
	Drift angle	Drift angle R<1/100	0°	1/147	28F	KOKUJI-HT	
Level 2			90°	1/155	14F	KOKUJI-KB	
	Ductility ratio	µ<2	0°	0.73	9F	KOKUJI-HT	
			90°	0.76	1F	EL Centro-NS	

Design seismic wave

Level 1;25kine, Level 2;50kine modified the configuration of maximum ground motion speed

KOKUJI- :Simulated ground motion waveform is created based on the spectrum defined in the Building Standards Law

integrated precast members (LR beams) in which the primary mode of column reinforcement is passthrough penetration holes made using sheath pipes, and precast columns (V columns) in which mortar-filled mechanical joints (sleeve joints) are built into the column heads from which column reinforcing bars protrude from the bottom and pass through the penetration holes in the beams. Mortar is used to fill in the joints between precast members, the main reinforcement pen-

etration holes and the LR beam-V column joints so that the respective members are integrated into the framing structure. (Fig. 2 and Photos 3~4)

Photo: Toranomon-Roppongi Work Office



Photos 3~4 V column and LR beam used in the LRV method

LRV-H Method

In the LRV-H (left tight vertical horizontal installation precast) method, two types of members are adopted: precast column members (H columns) that integrate columns and joints into a single structure and have a full length that is similar to the floor height, and precast beam members (H beams) that use the inner web depth as a member. In the column members, the penetration holes for primary reinforcement





(Source : Obayashi Corporation)

run horizontally. A sleeve joint is installed near the end of the intersecting beams, and the primary reinforcing bars extend to the end of the precast beam on the opposite side of the sandwiched column. The beam reinforcing bars pass through the beam reinforcement penetration holes provided in the column and are then inserted into the sleeve at the end of the beam end on the other side. (Fig. 2)

Hybrid Structure (Reinforced Concrete-Steel Beam Structure)

In order to realize column-free spaces on the office floors, steel girders were adopted for the long-span sections, and a joining method was adopted in which the steel girders are wrapped with concrete (Fig. 3). In the design stage, performance confirmation testing was conducted on the hybrid structure, the results of which are reflected in the design.

The yielding position of the beam was set inside the steel framing, which allows for the framing property to be tough even after yielding.

Brake Dampers

The brake dampers are structured like automotive disc brakes. The vibration energy of the building is converted to friction heat due to the damper's sliding under certain loads so that the response and damage of the building are mitigated. The brake damper system can be used repeatedly and requires no need for maintenance. (Fig. 4)







Fig. 3 Partial Steel Girder-Reinforced Concrete Hybrid Structure

Latest High-rise Project

Loop Road No. 2 Project —Super High-rise Building on Urban Highway—

by Hiroshi Takahashi, Nihon Sekkei, Inc.

The "Loop Road No. 2 – Shimbashi / Toranomon Redevelopment Project, Zone III" is a large-scale urban redevelopment project proposed for the Toranomon area of Tokyo. A notable feature of this project is that the planned Loop Road No. 2 will cross east to west through the development site and under a yet-to-be-completed super high-rise building with a height of 247 m. (Refer to Figs. 1 and 2)

The planned building is to be erected on a site of about 17,000 m², will have 52 floors above ground and five below ground, and will have a total floor area of about 244,000 m². As a mixed use building, the parking lot takes up the floors below ground, shops and a conference facility occupy floors 1 to 5 of the lower level, and offices fill floors 6 to 35 of the medium-rise section. Beyond this, the 36th floor is totally reserved for use as a space truss for a different arrangement of columns on the 37th floor and above. In the high-rise section, residential facilities are located from the 37th to the 46th floors, while a hotel occupies the 47th floor and above.

Most of the redevelopment zone is occupied by the super high-rise building and the underground section through which a tunnel for Loop Road No. 2 passes. A 3-story commercial building is located at the site, and a large freestanding eave that protrudes from the 2nd floor of the highrise building overhangs the plaza beside the high-rise building.

Outline of Building Structure

The aboveground section of the super highrise building is mainly a rigid steel frame structure (using concrete-filled steel tube columns) with response-control devices attached, and the commercial building and large eave are also steel frame structures. The underground section is a mixed structure composed of steel framing, steel frame-reinforced concrete, and reinforced concrete. The podium structure (artificial ground) on the Loop Road tunnel consists of 1 m-thick precast slabs.

As for construction of the foundation, because the inverted construction method

Fig. 1 Perspective Drawing of Loop Road No. 2 Project



was adopted in order to reduce the construction term, cast-in-place piles were used to form a piled raft foundation in which the piles and spread foundation bear loads respectively according to their rigidity.

Response-control Structure

A response-control structure was selected for the aboveground portion of the highrise section, which has a plane shape of $85 \text{ m} \times 61 \text{ m}$, and response-control devices are positioned in the various sections of the center core. In order to create a mega-frame that could effectively suppress bending deformation of the entire building, the entire structure from 1st story through the structureswitching truss up to the structureswitching section of the 51st story that adopts a combination of inclined roofing

Fig. 3 Floor Framing of Standard Floors



Fig. 2 Arrangement of Various Structures

and shifted roof tops to form a distinctive roof frame—is linked employing response-control devices (Fig. 3).

A combination of three kinds of response-control devices is used: oil dampers (516), buckling-restraint braces (448) and friction dampers (620). A response-control mechanism that capitalizes on this kind of combined use can successfully secure high seismic resistance by reducing the inter-story drift angle during an earthquake to 1.5-more times less than that of common high-rise buildings. (Refer to Fig. 4)

Connection for Tilted Column-crossed Section

The tilted column system is adopted for the northwest, southeast and southwest corners of the building for the following reasons: columns cannot be positioned above the area covering Loop Road No. 2; and the



corners of the building must be finished with sharply acute angles for aesthetic purposes. From the 8th to the 13th floors, two tilted columns on each upper floor intersect and combine to form a single column on the floor below. Cast steel connections, each weighing about 20 tons, are adopted for where the columns intersect so that the stress carried by the two upper columns can be securely transferred to the single column below. (Photo 1)



Photo 1 Cast steel connection for peripheral tilted column-crossed section

Countermeasures against Road Vibration

Running underground, Loop Road No. 2 connects on the east side of the building, at the second-basement level, with an underground tunnel installed outside the development site, and then resurfaces on the west side of the site. The road runs through a tunnel-shaped culvert structure that was built independently of the building structure. In order for road traffic

Fig. 5 Road Culvert Structure and Underground Building Structure



vibration not to affect the building, the building structure is insulated from the culvert structure by sandwiching vibration-damping material (polyurethane elastomer) between the culvert bottom slab lower section and the building structure. (Fig. 5)

Large Eave

A large freestanding eave is installed on the plaza at the west side of the building. It has skylights and measures about 57 m \times 32 m in outer dimension. Having an oval shape, it stands at a height of about 22 m above the podium structure. (Fig. 6)

The inner frame section, which is composed of skylight glass, adopts a singlelayer frame. In order to carry the inner frame and to maintain its own high rigidity, the frame is designed as a space truss composed mainly of steel pipes.

Fig. 6 Structural Frame for Large Eave





Photo 2 Construction of structure-switching truss



South-north section



Photo 3 Entire view of Loop Road No. 2 Project underway (November 2012)

Construction Is Steadily Proceeding

After demolition of the existing buildings, construction of the buildings planned in the project began in April 2011. The aboveground steel framing is being installed simultaneously with construction of the underground structure, using the inverted method. Construction of the structure-switching floor, a difficult section in the construction of the aboveground structure, has been successfully completed (Photo 2), and preparations are underway to build the complex framing at the top of the building.

Construction of the road culvert has started and is proceeding simultaneously with the high-rise building work. After completion of the high-rise building structure, extensive construction work will begin on the large eave, the commercial building and the podium structure. The grand opening for the entire project is scheduled in 2014. (Photo 3)

Latest High-rise Project

Otemachi 1-6 Plan – Ultra High-rise Building Using World-class Ultra High-strength CFT Columns–

by Shuichi Matsumoto and Shinichiro Kawamoto, Taisei Corporation

Two requirements have increasingly emerged in recent high-rise building construction projects: longer spans to enhance freedom in the space design of standard floors and wide atriums that promote structural continuity between the lower floors and the surrounding external space. In order to realize these two requisites, it is generally necessary for the structural members used in framing to bear large forces and, as a result, to grow in volume. However, the appropriate combination of highstrength members can prevent excessive increases in member volume.

As a structural technology to cope with such a task, we have developed worldclass, ultra high-strength, concrete-filled tubular (CFT) columns manufactured by the combined use of high-strength concrete (design standard strength: 150 N/mm²) and high-strength steel products (tensile strength: 780 N/mm²). These ultra highstrength CFT columns are being used to construct the low-rise floors of the ultra high-rise building in the Otemachi 1-6 Plan.



Photo 1 Building appearance

Building and Structural Outlines

This ultra high-rise building has 38 stories above ground (including a three-story penthouse), stands 199.7 m tall and has six basement floors extending to a depth of 35.1 m. The total floor space is about 198,000 m². (Photo 1). The main building functions are: offices, a hotel and shops.

In terms of structural type, the underground floors are built with reinforced concrete, and the aboveground floors form a steel structure that uses CFT columns for the 1st through 32nd floors. The aboveground floors form a moment resisting frame structure with response-control braces arranged at the core of the building. The oil dampers that are used as viscous dampers and the buckling-restraint braces that employ low yieldpoint steel (LY225) for the axial members and are used as hysteresis dampers are appropriately arranged so that seismic energy can be effectively absorbed.

Mega-truss frames are adopted for the 4th and 32nd floors. These are structurechanging floors in which the structural position of the columns changes in order to realize a framing system that allows different floor spans on the upper and lower floors. In order to secure adequate habitability during strong winds, a response-control device (active mass damper) is installed on the rooftop. (Refer to Fig. 1)

Application of Ultra High-strength CFT Columns

A unique feature of the Otemachi 1-6 Plan is the creation of Otemachi Forest, a verdant 3,600-m² area of soil and greenery provided to the ground floor of the build-

Fig. 1 Framing Elevation



Fig. 2 Combined Use of High-strength Steel and High-strength Concrete for Ultra High-strength CFT Columns



ing site. In this regard, an important task in the building plan was how to build safe structural framing with no structural hindrances in the large space provided between Otemachi Forest and the subway concourse below.

The ultra high-strength CFT columns applied in the low-rise section of the building were produced using 780 N/mm² steel and Fc150 N/mm² concrete, which give the columns world-class strength (Fig. 2). Adoption of these ultra high-strength CFT columns in the low-rise section satisfies the above-mentioned task in the building plan. In addition, these columns meet the requirements for seismic safety (Fig. 3). The elevated strength of these CFT columns suppresses the need to increase of the volume of the column members, which in turn leads not only to a reduction in the use of structural materials but also to the mitigation of associated environmental burdens.

Fig. 4 shows the relation between strain and the stress applied to the steel and concrete. The critical strain is nearly identical for both the 780 N/mm² steel and the Fc150 N/mm² concrete, which indicates the great advantage to be obtained by combining the use of high-strength concrete with high-strength steel. In addition to the extremely high vertical supporting capacity that these ultra high-strength CFT columns possess, they also exhibit a high yield ratio and low elongation at the time of tensile strength, thereby offering sufficient allowance to keep the building frame within its elastic range.

In the development of these ultra highstrength CFT columns, structural performance was confirmed by means of structural testing, and, further, the structural design work was conducted using the strength assessment method. In addition, other extensive testing was conducted,



Photo 2 Ultra high-strength concrete-filled tubular (CFT) column







such as welding tests supposing both factory and on-site welding of the ultra highstrength steel and both material and filling confirmation tests on the ultra highstrength concrete. Then, after the structural efficiency of these ultra high-strength CFT columns was confirmed, the columns were put into practical use.

Wide Floor Space Using Fewer Columns

Still under construction, the building will have a height of 199.7 m. Adoption of ultra high-strength CFT columns will successfully produce large, wide spaces with fewer columns on the lower floors that will structurally share the large burden associated with ultra high-rise buildings. (Refer to Photo 2)

Reference

 Shuichi Matsumoto, Tsutomu Komuro, Hiroyuki Narihara, Shinichiro Kawamoto, Osamu Hosozawa and Koji Morita: Structural Design of an Ultra High-rise Building Using Concrete Filled Tubular Column with Ultra High Strength Materials, 15th World Conference on Earthquake Engineering (15WCEE), Lisbon, Portugal, 2012.

JSSC Operations

I JSSC Symposium 2012 on Structural Steel Construction

The Japanese Society of Steel Construction (JSSC) has annually held its JSSC Symposium on Structural Steel Construction since 2004. The major aim of this symposium is to comprehensively and functionally link operating results of JSSC's various committees and working groups and to provide a venue for exchanges between JSSC members and those working in steel construction. The 2012 symposium was held on November 15 and 16.

The 2012 symposium covered a wide range of events such as those shown in the attached table, and also the introduction of the 2012 JSSC prize-winning works using panel exhibition. (For the prize-winning works, refer to pages 1~6.)

The total number of participants for the twoday 2012 symposium was about 1,000. This annual symposium serves as a useful venue for exchange between researchers and engineers working in steel construction and for colleting the latest information in steel construction.

The 2013 symposium is scheduled to be held in Tokyo on November 14 and 15, 2013.

To Our Readers

Toshiyuki Sugiyama Chairman International (

Chairman, International Committee, JSSC (Professor, Graduate School of Yamanashi University)



I assumed the position of chairman of the International Committee in 2012. Starting with issue No. 26 of Steel Construction Today & Tomorrow, our International Committee has been

responsible for the editorial planning of one of the three issues published annually. Since its inauguration, JSSC has conducted wideranging activities in the form of survey, research November 15, 2012

\setminus	General pro	ogram	Academy session			
A M	Special stainless steel session			Connection/Joint I	Loading capacity	
		Lecture by structural diagnosis consultant		(building construction)	(civil engineering)	
ΡM	Engineering session (Standadization Committee)	Awarding of JSSC Prize Awarding	Material (building construction)	Connection/Joint II (building construction)	Corrosion/Corrosion protection (civil engineering)	
	Special lecture I	ceremenony Lecture by prize winner	Hybrid and composite structures (civil engineering,	Structural member I (building constrcution)	Connection/Joint (civil engineering)	
		Special lecture II	building construction)			
	Social gathering					

November 16, 2012

	A M	International Committee meeting	Fatigue/Fracture (civil enginnering)	Vibration/Response constrol/Seismic resistance (civil engineering)	Structural member II (building constrcution)
	P	ISIJ-JSSC exchnage planning liaison group	Measurement/ Inspection/Monitoring (civil engineering)	Vibration/Response constrol/Seismic resistance (building construction)	Structural member III (building constrcution)
M	Construction"	Repair/Reinforcement (civil engineering)	Bridge (civil engineering)	Framing (building construction)	

and technological development aimed at promoting the spread of steel construction and at improving associated technologies and, further, to extend cooperation to related organizations overseas.

Following the merger of JSSC with the Stainless Steel Building Association of Japan in 2010, JSSC's field of operation has expanded to include not only carbon steel but also highly corrosion-resistant stainless steel. Consequently, we intend to actively transmit information throughout the world that is related to a wider range of steel construction areas.

As was true in issue No. 35, the previous issue for which our committee was responsible, our current issue, No. 38, introduces the outstanding works that received JSSC Prizes and Thesis Prizes. A special feature of this issue treats advanced high-rise building technologies in Japan and the latest high-rise projects. Also

included is an outline of JSSC Symposium 2012 on Structural Steel Construction.

The International Committee, while working on multi-faceted responses to the internationalization of steel construction specifications and standards, promotes exchanges of technical information and personnel between Japan and overseas organizations. As one aspect of these operations, we are attempting with this issue to inform our readers of JSSC operations, trends in steel construction, and the technologies and technological developments relevant to the planning, design, and building of steel structures in Japan.

If you wish to obtain more detailed information about the various articles contained in this issue or to receive related technical information, please do not hesitate to contact JSSC staff member Hiroshi Sugitani (h.sugitani@jssc.or.jp)

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