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Development of Building Structural Steel with High Yield Ratio and High Yield Point Leading to Innovative Steel Structural System

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Abstract

Damage control design of building structures to absorb seismic energy using seismic dampers to prevent damage to beams and columns has come to be widely applied to high-rise buildings. By this design method, the behavior of columns under seismic forces remains substantially within the elastic region, and naturally, the mechanical properties required for the steel used for columns should be different from those for beams. However, the same steel has been used conventionally for both the applications. In view of this, setting a target yield point higher than those of similar conventional steels, Nippon Steel has developed a new steel for building column use. This paper reports the parameters of the development, specifications and performance of the new steel.

1. Introduction

After the introduction of the New Earthquake-Resistant Design Code in 1981 under the revised Building Standard Law and the quality problems of building frames in the 1990s, the properties that steel materials for building structural use should have attracted wide attention. The desired performance of structural steels was studied in an effort to meet the requirements for the design and building aspects. Accordingly, a new grade of rolled steel for building structural use (SN) was included in the Japanese Industrial Standards (JIS) system in 1994; the specifications for weldability (carbon equivalent C_{eq}) and seismic resistance (low yield ratio YR, narrow range of yield point YP, and Charpy absorbed energy $_{v}E$) were newly included for the SN steels in addition to those of conventional SS (rolled steels for general structure) and SM (rolled steels for welded structure) steels. The inclusion of SN steels in the JIS system made the performance items clear that steels used for building structures in the country.

After the standardization of the SN steels, the Hanshin-Awaji Earthquake in 1995 caused building owners to attach more importance to the property value of buildings, and in view of this, the related industries have made efforts to enhance the seismic resistant reliability of building structures. More recently, an inter-ministerial project named the Development of New Building Structural System Using Innovative Materials started (under cooperation between the Cabinet Office, Ministry of Economy, Trade and Industry and Ministry of Land, Infrastructure and Transport) envisaging urban redevelopment and social capital improvement. A new building structure system, dramatically more seismic resistant and versatile than conventional ones, called the New Structural System is being developed under the framework of the project[1, 2].

The target of the New Structural System is to economically

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achieve an unprecedented level of seismic resistance to withstand a Scale 7 earthquake (Japanese scale) by elastic design of columns and beams. Here, an important development policy is functional division of structural members typically such as the "skeleton and infill" and "separate-horizontal-and-vertical-loads structure". The philosophy of different functions for different structural members inevitably requires new innovative structural materials, and the policy for the development of such new steel materials economically is to identify newly required property items, improve them in priority but drastically disregard those that are not so strongly required. Developing innovative steel materials excellent in specifically required property items and selectively applying them to different structural parts make the New Structural System viable.

In consideration of the movement as outlined above and as a precursory action for the future shift from the current building structure to the New Structural System, Nippon Steel Corporation has reviewed the current material standards based on the philosophy of functional division of structural members and use of specialized materials for different structural applications, and as a result, developed a new steel for building structural use.

2. Elastic Column Design and Development of Heavy Steel Plates for Column Use

2.1 Property items required for building structural steels

A moment-resistant-frame structure composed of columns of square hollow sections and beams of H sections accounts for about 90% of steel-framed buildings. The columns and beams are connected to each other rigidly using diaphragms, etc., and welding is widely employed for the connection at either fabrication plants or construction sites. Under large seismic force, the columns, beam ends and column-to-beam connections are allowed to deform plastically to absorb seismic energy and thus prevent the whole structure from collapsing (see **Fig. 1**). Detailed structural study for low-rise buildings under strong seismic force is omitted, as structural members are implicitly expected to deform plastically. Consequently, as shown in **Fig. 2**, weldability and seismic resistant properties are strongly required for steel materials for building structural use regardless of which structural members they are applied to.

In the meantime, a seismic energy absorbing device called a seismic damper came to be incorporated in the structural frames of high-rise buildings (see **Fig. 3**) since the early 1990s, and thus, damage control structural design[3)] to reduce the extent of plastic deformations of columns and beams became a common practice. After the Hanshin-Awaji Earthquake, the need for the higher asset values and prompt recovery of buildings after a heavy earthquake increased so much that virtually all the structural frames of high-rise buildings constructed thereafter have seismic dampers. A building frame by the latest design philosophy is composed of columns, beams and dampers having their respective functions, and as described below, the steels used for them must have different mechanical properties in accordance with the functions required for each of the structural members.

Dampers: Dampers are expected to deform plastically to absorb seismic energy before columns and beams do. The steel used for this application is required to have a low yield point, narrow range of yield point fluctuation and high elongation.

Beams: Beams are expected to deform plastically to absorb the portion of seismic energy exceeding the capacity of the dampers, and to yield before columns do to prevent them from being damaged. For these purposes, the steel used for beams must have a low yield ratio and narrow range of yield point fluctuation.

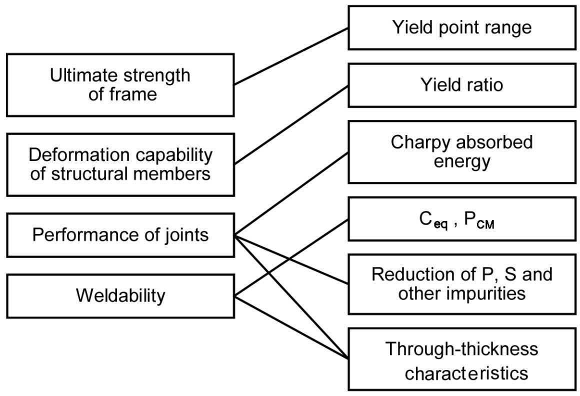

Fig. 2 Performance requirement for steel used in building structure

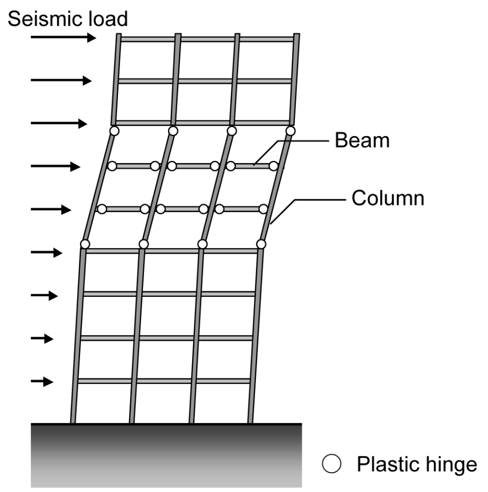

Fig. 1 Seismic design utilizing plastic deformation capacity of beam and column

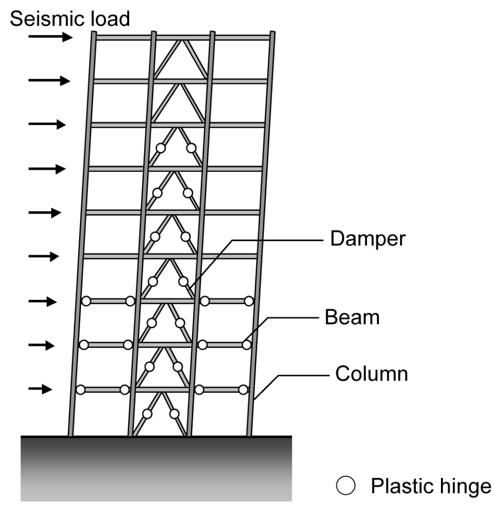

Fig. 3 Damage control design preventing column damaged

Columns: Columns support all the weight of the building to prevent it from collapsing under large seismic force. Protected by the two-stage energy absorption by dampers and beams, columns remain substantially within an elastic deformation region. Since the sum of the building weight and seismic load imposed on a column increases in lower stories, the steel used for this application must have a high design strength.

In consideration of these required properties, low-yield-point steels have been developed for damper applications and widely accepted in the market. For beam and column applications, on the other hand, in spite of significantly different plastic deformation behaviors required for them, the same grade of steel based on the JIS standard for SN steels is still being used for both. In view of the above and expecting that the damage control structural design will become widely practiced, the authors decided to propose a new steel grade different from the SN steels and having the characteristics and functions required for the columns in the damage control structural design.

2.2 Proposal of new steel grade for building column application

The height and column-to-column span of buildings are increasing, and as a result, increasingly thicker steel plates came to be used for building columns. Since increase in material thickness leads to increased structural weight and welding work, use of high-design-strength steels (for example, SA440, high-performance steel for building structural use of a 590-N/mm^2 class) is effective in controlling the increase in construction costs. It should be noted, however, that to obtain a high design strength of a steel while keeping its yield ratio at 80% or less, it is necessary to increase its tensile strength as well, which means increased addition of carbon and other alloying elements. However, increased use of alloying elements is detrimental to weldability, causing significant increase in hardness and cracking of heat-affected zones (HAZs) of weld joints. All these mean that, even if a low yield ratio is realized with a high-strength steel, it will not be able to fully exert its plastic deformation capacity because of easy occurrence of brittle fracture starting from a hardened or cracked portion.

To prevent this, stringent work control such as preheating is required for welding of SA440. As described above, design strength, yield ratio and weldability, in close interrelation with each other, exert influence on the properties of structural steels. These three property items are in a good balance in the SN steels, but when a design strength higher than that of the SN steels is attempted, the good balance between the three is difficult to maintain, and it becomes necessary to choose one of the two mutually incompatible property items, low yield ratio and high weldability.

To prevent a building from being destroyed by a strong earthquake and minimize the residual deformation of the whole structure to enable its reuse, it is essential to take design measures so that the behavior of columns remains within the elastic deformation region, which meets the latest need for safety and an enhanced asset value of a building. It is expected that such an elastic column design method would not be limited only to high-rise buildings but expand to a wider variety of buildings. This design method minimizes the required deformation capacity of columns, and it will be possible to relax the restriction on yield ratio. This will permit a higher yield point without having to change tensile strength from that of conventional steels (see **Fig. 4**), and accordingly, a higher design strength without sacrificing weldability. New high-yield-ratio, high-yield-point steel plates(BT-HT400C) for building structural use were developed based on this concept.

Table 1 compares the specifications set out for the development of the new steel with those of Nippon Steel's BT-HT325 and 355 (TMCP steels for building structural use), which have design strengths higher than that of the SN steels, and SA440; all the comparative steels have yield ratios of 80% or less. The range of yield ratio of the developed steel was expanded (relaxed) to 90% or less, and its design strength was set at as high as 400 N/mm^2, which is higher than that of BT-HT325 by 23% and close to that of SA440, while its tensile strength was in the same class as that of BT-HT325 (490 N/mm^2).

The chemical composition specified for the new steel was the same as that for BT-HT325, and indicators of weldability, namely the carbon equivalent (C_{eq}) and chemical composition on sensitivity of welding crack (P_{CM}), were set lower than those of BT-HT355 and SA440. As a result, plates of the new steel do not require preheating for normal welding in all the thickness range. In addition, since the developed steel was meant mainly for application to welded square hollow section columns, the upper limits of P and S were set lower, and the specification includes provisions for the homogeneity of material properties in the thickness direction and ultrasonic test, like

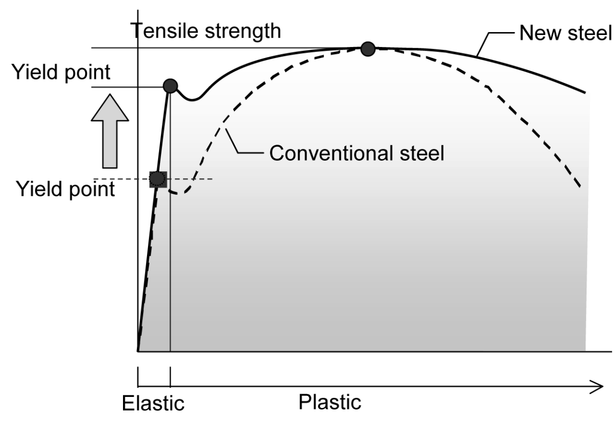

Fig. 4 Development concept of high yield point steel

Table 1 Comparison of specifications with conventional steel

Standards	Design strength (N/mm^2)	Thickness (mm)	Yield point (N/mm^2)	Tensile strength (N/mm^2)	Yield ratio (%)	C_{eq} (%)	$_vE_0$ (J)	f_{HAZ} (%)
BT-HT400C	**400**	**19-100**	**400-550**	**490-640**	**≦90**	**≦0.40**	**70≦**	**≦0.58**
BT-HT325	325	40-100	325-445	490-610	≦80	≦0.40	27≦	-
BT-HT355	355	40-100	355-475	520-640	≦80	≦0.42	27≦	-
SA440	440	19-100	440-540	590-740	≦80	≦0.47	47≦	-

in the case of Class C of the SN steels.

As regards to impact properties, assuming the standard welding condition of CO_2 gas-shielded arc welding (CO_2 welding), the chemical composition (more specifically, f_{HAZ}, an index of the toughness of HAZs by MAG welding[4]) was so defined that the Charpy absorbed energy of a HAZ at 0℃ was 70 J or more. This is because, to prevent brittle fracture at a welded beam-end joint designed to deform plastically, it is desirable to secure a Charpy absorbed energy of 70 J or more at 0℃[5]. In consideration of the risk of welded joints of building pieces and other fixtures serving as initial points of low-stress brittle fracture and the fact that welding work at construction sites is not always done under due control, it is important for steel for construction use to have good weldability and impact properties.

The plate thickness of the developed steel was set at 19 to 100 mm to make it possible to fabricate columns from the lowest floor to the building top using the same grade and class of steel. The development policy assumed that columns would be designed not to deform plastically up to an ultimate condition, and for this reason, the allowable ranges of yield point and tensile strength were set wider than those for conventional steels.

2.3 Production of plates of high-yield-ratio, high-yield-point steel

Plates were test produced in accordance with the specifications for the developed steel and their material properties were examined. The thermo mechanical control process (TMCP) was applied to the plate production, and a high yield point was obtained through accelerated cooling even though the chemical composition was the same as that of a 490-N/mm^2 class steel; a yield point of 400 N/mm^2, nearly that of SA440, was obtained while securing the same weldability as that of BT-HT325. The TMCP was originally developed to increase steel strength by forming a fine metallographic structure, but when applied to steels for construction use, this advantage of the process is abandoned to lower the yield point elaborately so as to obtain a yield ratio of 80% or less. The developed steel, in contrast, was produced applying the TMCP according to its original purpose to obtain a high yield point, which made it possible to raise the design strength of the plates at production costs comparable to those for conventional steels.

Photo 1 compares the microstructure of the new steel with that of conventional steel for building structural use. The crystal grains of the developed steel are fine, which raises the yield point and enhances impact properties of the base metal. The authors tested the material properties of the test-produced plates 25, 50 and 100 mm in thickness; the results are described below.

Table 2 shows the chemical composition of the specimen plates together with their values of C_{eq}, P_{CM} and f_{HAZ}, and **Table 3** the results of their mechanical test. Note that the plates 25 and 50 mm in thickness were rolled from the same heat of molten steel.

The stress-strain relationship at tensile test is shown in **Fig. 5**. The yield point and yield plateau tend to become unclear with increasing plate thickness, and the stress-strain curve assumes a round-roof shape; the tendency is the same with conventional TMCP steels. When yield point does not show clearly, 0.2% proof stress is used instead for guaranteeing the standard value. The yield ratio of the 25-mm thick plate was over 80% and those of the 50- and 100-mm thick plates were less than 80%.

Fig. 6 demonstrates the energy transition curves at Charpy impact test. The transition temperature ranged from –74 to –52℃, and

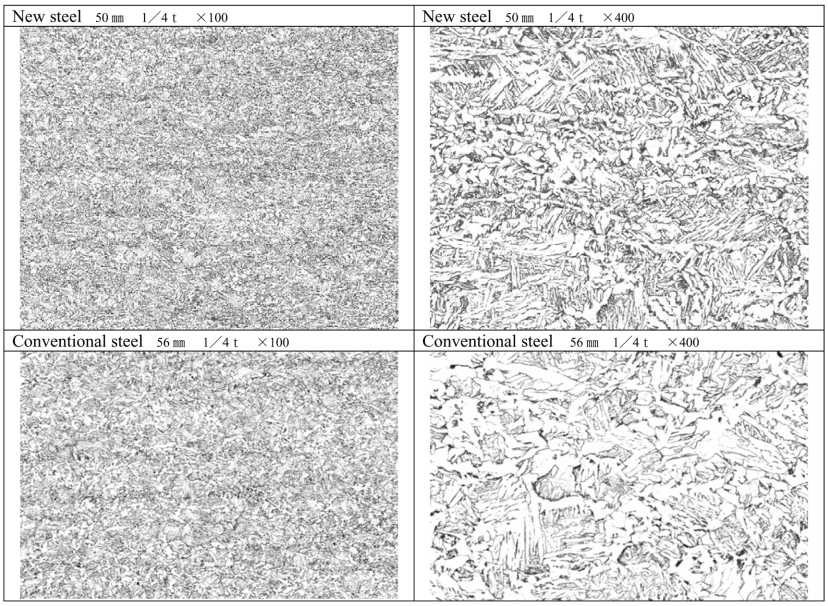

Photo 1 Micro-structure of conventional TMCP steel and new steel

Table 2 Chemical compositions of sample steel (mass %)

New steel	Thickness (mm)	C	Si	Mn	P	S	C_{eq}	P_{CM}	f_{HAZ}
Specification	19-100	≦0.20	≦0.55	≦2.00	≦0.020	≦0.008	≦0.40	≦0.26	≦0.58
Sample A	25	0.14	0.27	1.30	0.014	0.003	0.37	0.22	0.39
Sample B	50	0.14	0.27	1.30	0.014	0.003	0.37	0.22	0.39
Sample C	100	0.10	0.22	1.54	0.007	0.002	0.37	0.19	0.35

Table 3 Mechanical properties of sample steel

	Thickness (mm)	Tensile test: Yield point (N/mm²)	Tensile test: Tensile strength (N/mm²)	Tensile test: Elongation (%)	Tensile test: Yield ratio (%)	Charpy impact test: Absorbed energy at 0℃ (J)	Through thickness tensile test: Reduction of area (%)
Specification	19-100	400-550	490-640	21≦ *1 23≦ *2	≦90	70≦	25≦ (ave.) 15≦ (each)
Sample A	25	455	554	25 *1	82	311, 305, 315 (ave.310)	67, 62, 72 (ave.67)
Sample B	50	442	576	26 *1	77	316, 317, 321 (ave.318)	71, 71, 71 (ave.71)
Sample C	100	412	546	34 *2	76	327, 339, 337 (ave.334)	80, 78, 78 (ave.79)

*1: Gage length = 200mm *2: Gage length = 50mm

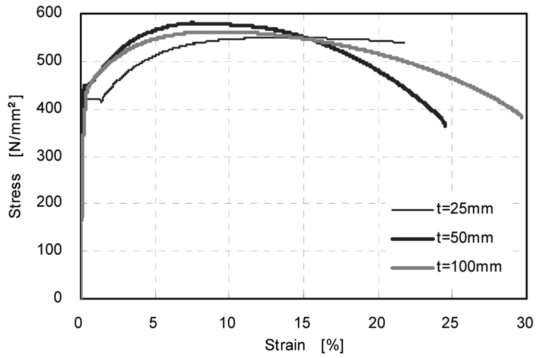

Fig. 5 Stress-strain curves from tensile test

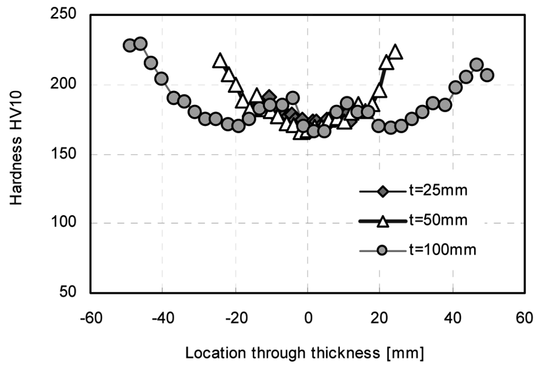

Fig. 7 Hardness distribution in direction of thickness in section

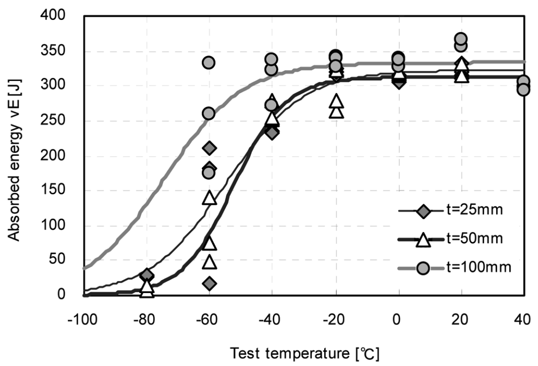

Fig. 6 Energy transition curves from Charpy impact test

at a test temperature of 0℃, all the readings of the specimens were on the upper shelf always exceeding 300 J, which evidences excellent impact properties of the developed steel due to the fine crystal structure as seen with Photo 1, better than those of conventional TMCP steels of low yield ratios.

Fig. 7 shows the results of sectional hardness test in Vickers hardness. The hardness fluctuation within a section surface increased with increasing plate thickness, the reading being higher nearer the plate surfaces. Substantially the same tendency is seen with conventional TMCP steels, and it does not adversely affect the structural performance of the material.

To confirm the weldability, the authors conducted maximum HAZ hardness test (JIS Z 3101) and y-groove weld cracking test (JIS Z 3158) using the 50-mm thick plates and by CO_2 welding. The test conditions and results are given in **Tables 4 and 5**. The maximum

Table 4 Test result of maximum hardness test in weld heat-affected zone

Thickness	Test method	Welding process	Welding material	Pre-heat	Heat input	HV10
50mm	JIS Z 3101	CO_2 *1	JIS Z 3312 YGW18	None	17 kJ/cm	253

*1: CO_2 gas-shielded arc welding

Table 5 Test result of y-groove weld cracking test

Thickness	Test method	Welding process	Welding material	Pre-heat	Heat input	Surface crack	Section crack	Root crack
50mm	JIS Z 3158	CO_2	JIS Z 3312 YGW18	None	17 kJ/cm	None	None	None

HAZ hardness (HV10) was 253 and no undesirable hardness change was observed. No cracking was observed in the y-groove weld cracking test, either, attesting to excellent weldability.

3. Performance of Building Columns of Welded Square Hollow Sections Using Plates of High-yield-ratio, High-yield-point Steel

3.1 Confirmation of performance of welded joints through welding test

3.1.1 Outlines of test

Since building columns of welded square hollow sections are mostly fabricated employing a large-heat-input welding method such as CO_2 welding, electro-slag welding (ESW) or submerged arc welding (SAW), the authors prepared actual-size joints of the 50-mm thick plates by each of the three methods and tested the mechanical properties of the joints and HAZs.

Table 6 shows the welded joint performance targeted at the test. The target minimum tensile strength of the welded joints and weld metal was set at 490 N/mm^2, equal to the tensile strength of the base metal. Assuming that the yield strength of the weld metal at a corner joint of the square hollow section by SAW and that of a column-to-column joint by CO_2 welding should be equal to or higher than that of the base metal, the target yield strength of the weld metal of these joints was set at 400 N/mm^2. On the other hand, with respect to the inner diaphragm welding by ESW, supposing that the material of beams would be of the TMCP325 class and that it would be enough for a joint to have a yield strength equal to that of the beam, the target yield strength of the weld metal was set at 325 N/mm^2. The target value of Charpy absorbed energy at 0℃ (notch toughness) was set at 70 J for CO_2 welding, corresponding to f_{HAZ} of the base metal, and those for large-heat-input SAW and ESW were set at 27 J, equal to the notch toughness of conventional TMCP steels. The target maximum hardness of HAZs was set at 350.

3.1.2 Welding conditions

The welding conditions for the test were defined such that the heat input would be as large as possible within the range of common steel structure fabrication. **Fig. 8** shows groove configurations, and **Table 7** the condition for each of the welding methods. In the case of diaphragm welding by ESW, plates of the same steel grade and thickness (50 mm) as those of the column skin plates were used for the inner diaphragms, and as a result, the heat input was nearly 750 kJ/cm. In the case of SAW, skin plates 50 mm in thickness were welded in one pass by single-layer welding using two electrodes.

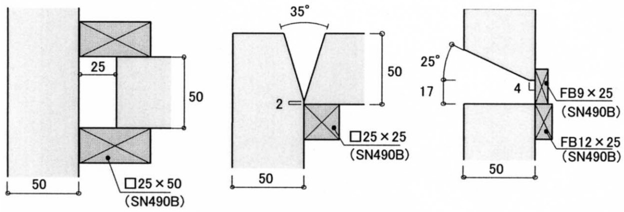

Fig. 8 Groove configuration of welded joints

The welding consumables were selected in consideration of the target performance of the welded joints. To allow a relaxed welding condition and secure a weld metal toughness of 70 J, YGW18 (JIS Z 3312) was selected for CO_2 welding. In consideration of the decrease in yield strength of weld metal under a large heat input, S582-H (JIS Z 3183) of the 590-N/mm^2 class was selected for SAW. For ESW, YES51 (JIS Z 3353), conventionally used for 490-N/mm^2 class steels, was selected because it was enough for welded joints to have the same yield strength as that of beams. No preheating was done in any of the welding methods.

3.1.3 Test results

Fig. 9 shows the notch positions of Charpy impact test pieces, **Fig. 10** photographs of macrostructures and hardness distribution graphs of the welded joints, and **Table 8** the results of their tensile and Charpy impact tests. The macro photographs attest that there are no welding defects and the welded joints had a sufficient width and good shape of penetration.

The tensile test of the welded joints was conducted using uniform gauge test pieces prepared by removing excess weld metal and backing plate by machining, and the same of the weld metal using round bar test pieces cut out from the 1/4 thickness position. All the test pieces cleared the target values. All the tensile test pieces of the welded joints failed at a HAZ of the base metal, which coincided with the softened portion explained herein later.

In the case of the diaphragms welded by ESW, hardness was measured at the thickness center, and in the cases of plates welded by SAW and CO_2 welding, at 2 mm below the plate surface. While

Table 6 Targeted level of welded joint performance

	Welding process	Yield strength of weld metal	Tensile strength of weld metal	Tensile strength of welded joint	Notch toughness*3	Maximum hardness
Inner diaphragm	ESW*1	325≦	490≦	490≦	27≦	≦350
Corner joint	SAW*2	400≦	490≦	490≦	27≦	≦350
Column to column	CO_2	400≦	490≦	490≦	70≦	≦350

*1: Electroslag welding *2: Submerged arc welding *3: Charpy absorbed energy

Table 7 Welding conditions

Welding process	Welding consumable	Welding pass	Current (A)	Voltage (V)	Welding speed (cm/min)	Heat input (kJ/cm)
ESW	JIS Z 3353 YES51	1	380	52	1.6	741
SAW	JIS Z 3183 S582-H	1	1900/1500*1	40/45*1	18	485
CO_2	JIS Z 3312 YGW18	1-44	230-280	28-32	19-65	≦40

*1: Twin electrode leading electrode/trailing electrode the HAZs of SAW and ESW exhibited a tendency to soften, and in contrast, those of CO_2 welding another to harden, all the readings satisfied the target hardness of HV10 $\leqq$ 350. Judging from this and the fact that all the welded joints cleared the target values for the tensile test without showing any sign of brittle fracture, the change in hardness will not adversely affect the structural performance of the steel frame. A similar tendency has been observed with conventional TMCP steels as well.

As is the case with conventional building structural steels, the toughness of HAZs tended to lower at the Charpy test with higher welding heat input. The column-to-column joints by CO_2 welding exhibited absorbed energy values significantly above the target value of 70 J corresponding to the specification of f_{HAZ}. The Charpy absorbed energy of all the SAW joints cleared the target value of 27 J although some of the readings were lower than the specification value for the base metal of 70 J. The absorbed energy tended to be lower at positions away from a fusion line (FL); this is presumably because the positions of these test pieces were nearer the thickness center and they were affected by center segregation. The absorbed energy of ESW joints was roughly 70 J when the heat input was 740 kJ/cm, and the amount of penetration under this condition was roughly 3 mm on one side. However, one must be careful about increasing heat input to secure sufficient penetration because, if excessive, it leads to lowering of HAZ toughness.

It should be noted that the above Charpy test results of the SAW and ESW joints are substantially at the same level as those of conventional steels. There are still various discussions about how good impact properties a welded joint must have to withstand possible destructive force and further clarification of the issue is awaited. While measures from the design and construction viewpoints are important for preventing destruction of structures, application of technologies

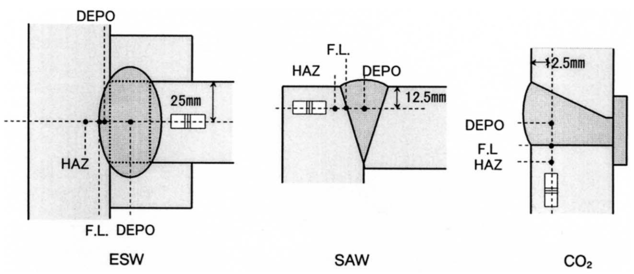

Fig. 9 Location of Charpy impact test specimens

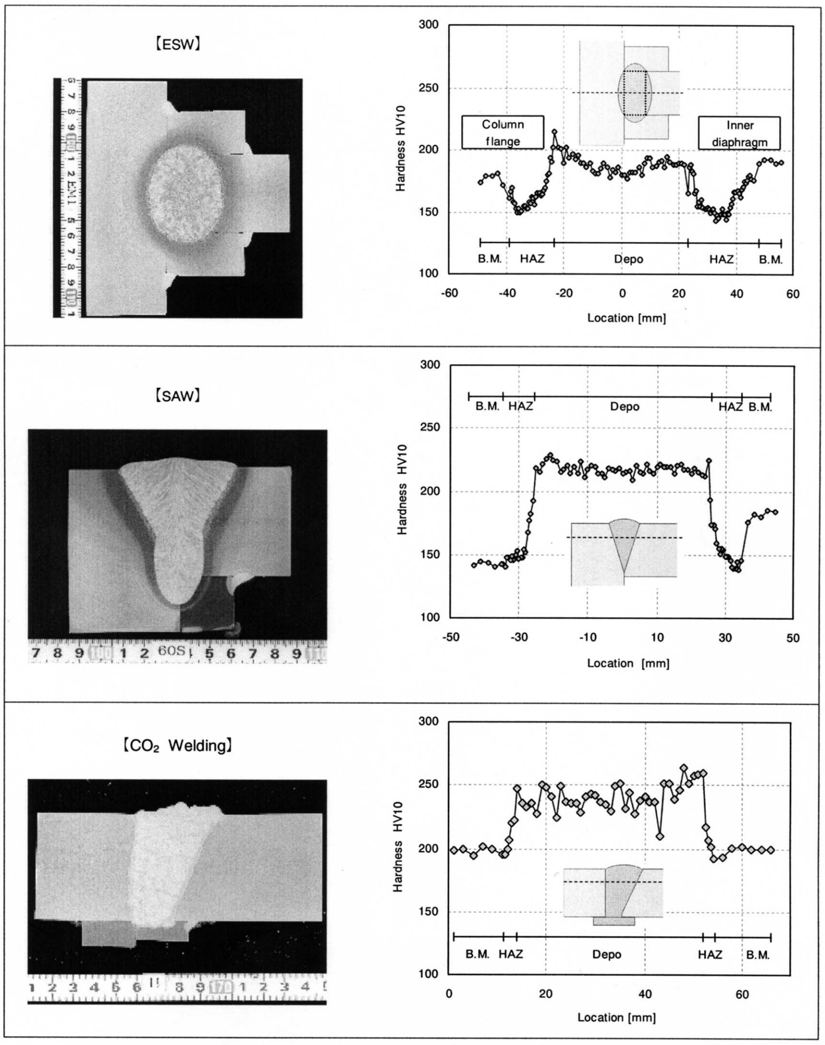

Fig. 10 Macro-appearance and hardness test results of welded joints

Table 8 Results of tensile tests and impact tests

Welding process	Test of weld metal		Test of welded joint		V-notch Charpy impact test			
	Yield strength*1	Tensile strength*1	Tensile strength*1	Location of fracture	Absorbed energy (J) (average of three tests)			
ESW	364	530	-	-	Depo*2	70	F.L.+1mm	89
	362	526			Depo*3	184	F.L.+3mm	110
					F.L.	140	F.L.+5mm	145
SAW	472	674	546	Base metal	Depo*2	97	F.L.+1mm	61
	480	678	553	Base metal	F.L.	71	F.L.+3mm	57
							F.L.+5mm	35
CO_2	653	701	603	Base metal	Depo	139	F.L.+1mm	240
	682	696	613	Base metal	F.L.	198		

*1: N/mm² *2: Center of weld metal *3: Edge of weld metal

to improve HAZ toughness (typically such as HTUFF®) [6] is effective from the viewpoint of materials.

3.2 Confirmation of column performance through compression Test

While columns are expected to behave within an elastic region under foreseeable external force, it is necessary to confirm how they fail ultimately under force larger than foreseeable. The deformation capacity of a structural member decreases with increasing yield ratio, and for this reason, when steel with a high yield ratio is used for building columns, it becomes important to ensure their elastic behavior through measures such as keeping the column overdesign factor within a prescribed range.

With a structural member under permanent compressive axial force such as a building internal column, however, stress fluctuates mainly in the compressive range, and the member finally fails by local buckling. The occurrence of local buckling is influenced considerably by width-thickness ratio; actually some papers report that the level of yield ratio does not significantly affect deformation performance[7]. The authors confirmed through tests of the developed steel that, in the case where maximum strength was determined ultimately by local buckling, it was possible to expect a steel having a high yield ratio and yield point to bear in the same manner as that of conventional low-yield-ratio steels.

3.2.1 Stub-column test

To clarify the relationship between the local buckling behavior and yield ratio of a square hollow section, the authors conducted stub-column test. **Tables 9 and 10** show the chemical composition and mechanical properties of the steel used for the test. Note that, because of the limited capacity of the test facility, the 12-mm thick plates, outside of the BT-HT400C specification, were prepared and used.

Five column specimens were fabricated using plates of the same thickness; the width-thickness ratio was changed by changing the outer dimension of the specimens, and their height was set at three times the outer dimension. The corner joints were welded by full-penetration CO_2 welding, and the excess weld metal was removed after the welding work. As shown in **Fig. 11**, a specimen was placed between the loading and bearing plates such that the load was applied evenly onto its upper end face, and the axial deformation of the specimen was measured using displacement gauges provided between

Table 10 Mechanical properties of sample steel using structural experiment

Thickness (mm)	Yield point (N/mm²)	Tensile strength (N/mm²)	Elongation*1 (%)	Yield ratio (%)
12.4	439	538	23	82
11.1	439	531	23	83
9.1	437	527	24	83

*1: Gage length = 200mm

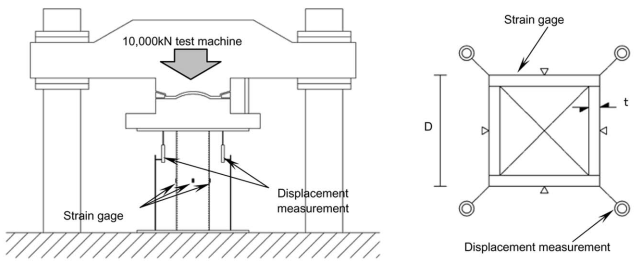

Fig. 11 Set-up of stub-column test

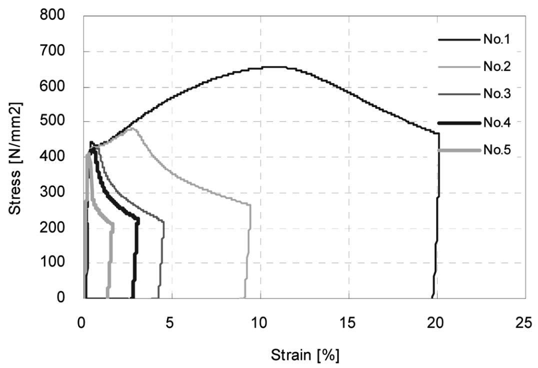

Fig. 12 Stress-strain curves obtained from stub-column test

Table 9 Chemical compositions of sample steel using structural experiment (mass%)

Thickness	C	Si	Mn	P	S	C_{eq}	P_{CM}	f_{HAZ}
12mm	0.12	0.24	1.41	0.010	0.002	0.37	0.20	0.37

Table 11 Results of stub-column test

	D/t	σ_{max} (N/mm²)	ε_{max} (%)	$\tau = \sigma_{max}/\sigma_y$	$\mu = \varepsilon_{max}/\varepsilon_y$
No.1	9.6	655	10.50	1.57	51.8
No.2	17.4	480	2.71	1.15	13.3
No.3	24.1	426	0.69	1.02	3.4
No.4	27.0	422	0.55	1.01	2.7
No.5	34.7	410	0.28	0.98	1.4

D, t : Width and thickness of stub-column specimen
σ_{max} : Maximum strength obtained from stub-column test
ε_{max} : Strain at σ_{max} obtained from stub-column test
σ_y : Yield strength obtained from tensile test
ε_y : $=\sigma_y/E$ (E : Young's modulus)

the loading and bearing plates.

Fig. 12 shows the stress-strain relationship thus obtained, and **Table 11** the test results. The symbol σ_{max} indicates the maximum stress, and ε_{max} the strain under σ_{max}. All the specimens recorded decrease of the strength when local buckling occurred; specimen No. 1, which had the smallest width-thickness ratio, buckled in an accordion fashion, and exhibited higher strain and stress than the others did.

3.2.2 Beam-column test

In addition to the above, using the equipment shown in **Fig. 13**, the authors conducted beam-column test, wherein cyclic shear force was applied to the center of a column specimen kept under constant, compressive axial force. Four specimens with width-thickness ratios D/t ranging from 18 to 36 were prepared, and the axial force ratio (ratio of the force imposed on the specimen to its yield axial force, which is yield point × sectional area) was set at 0.3. The length of the specimens was so defined that their slenderness ratio was roughly 20. A loading plate was provided at the length center of each specimen in the form of a through diaphragm. **Table 12** shows the specifications of the specimens; the 11.1- and 9.1-mm thick plates were prepared by machining the 12.4-mm thick plates from both sides. The cyclic shear loads were applied in the following manner: first, positive and negative loads one half the yield strength were applied in two cycles; then positive and negative displacements twice δ_p were applied in two cycles, δ_p being the deformation under the full plastic moment; then 4 times δ_p, 6 times δ_p, and so forth until the specimen failed.

The specimens underwent local buckling and recorded the maximum moment strength as follows: specimen No. 1 during the load shift from – 4 times δ_p to + 6 times δ_p; specimens Nos. 2 and 3 during the cycle of ± twice δ_p; and specimen No. 4 when the loading was nearly + twice δ_p. **Fig. 14** shows the relationship between the bending moment and rotation angle of the specimens. The bending moment was calculated in consideration of the additional bending resulting from the compressive axial force. The test results are listed in **Table 13**, where M_{max} is the maximum moment strength, η is the cumulative plastic deformation factor, and θ_{max} is the deformation under M_{max} calculated from a skeleton curve.

3.2.3 Discussion on test results

The authors compared the results of the stub-column and beam-column tests described above with those of similar tests using specimens of conventional steels (400 to 590-N/mm² class steels with yield ratios of 70 to 90%)[7]. Here, the results of the stub-column test were compared in terms of the strain-plasticity ratio calculated by dividing the strain under the maximum stress by yield strain, and the results of the beam-column test in terms of the ductility factor η of the skeleton curve by δ_p. In consideration of different strengths of

Table 12 List of beam-column specimens

	D (mm)	t (mm)	D/t	L (mm)	N/N_y
No.1	216	12.4	17.5	1 400	0.3
No.2	300	12.4	24.4	2 000	0.3
No.3	308	11.1	27.7	2 000	0.3
No.4	324	9.1	35.7	2 000	0.3

L : Length of stub-column specimen
N : Applied compression load
N_y : $=\sigma_y A$ (A: section area)

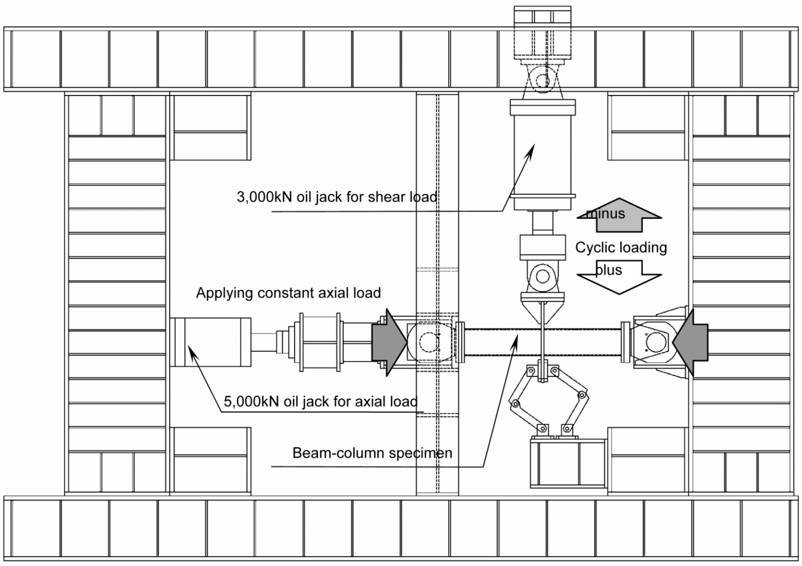

Fig. 13 Set-up of beam-column test

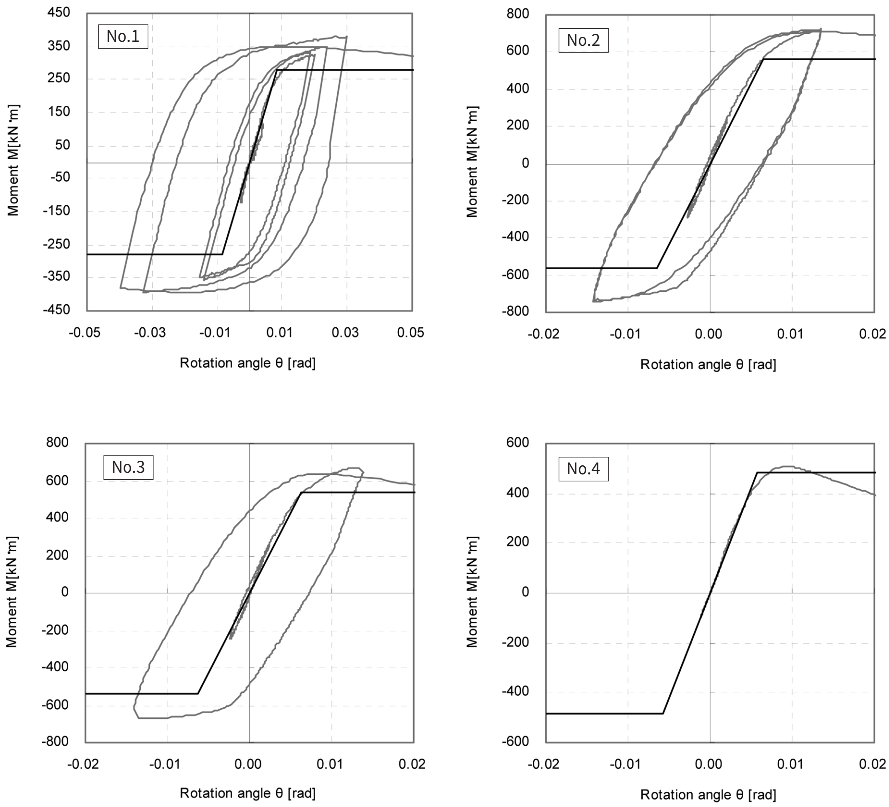

Fig. 14 Results of beam-column test

Table 13 Results of stub-column test

	M_{max} (kN•m)	θ_{max} (rad)	$\mu = \theta_{max} / \theta_p$
No.1	391	0.05	6.6
No.2	693	0.02	3.1
No.3	619	0.02	3.2
No.4	508	0.01	1.6

M_{max}: Maximum moment strength obtained from beam column test
θ_{max} : Rotation angle at M_{max} on skeleton curve obtained from beam column test
θ_p : Elastic rotation angle corresponding to M_p
M_p : Full plastic moment of specimen considering axial load

different steels, the comparison was done in terms of the equivalent width-thickness ratio $(D/t)_{eq}$ expressed by equation (1). **Fig. 15** shows the relationship between the equivalent width-thickness ratio and ductility factor.

$$(D/t)_{eq} = (D/t)\sqrt{(\sigma_y / E)} \qquad (1)$$

The test results of conventional steels indicate the following: buckling behavior depends mainly on the width-thickness ratio; when the ratio is large, buckling occurs soon after yielding, which leads to out-of-plane deformation before the stress of the steel material increases, and as a result, buckling behavior changes only slightly depending on yield ratio; and only when the width-thickness ratio is small, the ductility factor tends to increase slightly as the yield ratio decreases. It is clear from Fig. 15 that the developed and conventional steels demonstrate substantially the same tendency, which indicates that the deformation behavior of the developed steel under compressive force can be evaluated using substantially the same method as that for evaluating the conventional low-yield-ratio steels.

4. Summary

As a result of recent use of seismic dampers and introduction of a design method whereby building beams are made to yield before columns do, columns are made to remain in an elastic region even under strong seismic force. In consideration of this recent trend, the authors initiated the idea of a new high-yield-point and high-yield-ratio steel for building column application giving due consideration to higher design strength and improved weldability, test produced heavy plates of the new steel, fabricated welded square hollow sections and confirmed the performance of the developed steel when applied to building columns.

As the design strength of building structural steel increases, it is possible to restrict the increase in the skin plate thickness of square-hollow-section columns and decrease steel weight and welding volume. In addition, higher design strength expands the region where building columns behave elastically and increases the rigidity ratio between seismic dampers and the main building frame, which makes it easier for the dampers to show their effect. In designing the chemistry of the new steel, the authors allowed a higher yield ratio than those of conventional structural steels and aimed at enhancing weldability, and as a result it became possible to lower the costs for welding work and improve the quality and performance of welded joints. Another advantage of the developed steel is that, depending on heat input and inter-pass temperature, it can be welded using com-

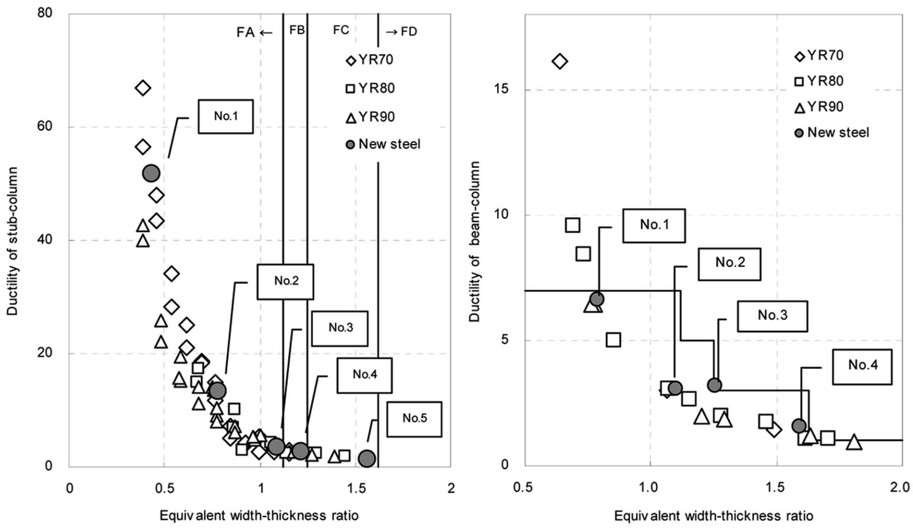

Fig. 15 Relationships between ductility and width-thickness ratio

mon welding consumables for 490-N/mm^2 class steels.

When applying the developed steel, it is essential to confirm that the columns will behave substantially within an elastic region under the design conditions. More specifically, this will be done, directly, by seismic response analysis or, more indirectly, by securing a column overdesign factor that would guarantee a overall collapse mode.

On the other hand, one may point out the possibility of the behavior of building columns going far beyond an elastic region and becoming plastic because of things such as unexpectedly large seismic force, fluctuation of material properties and inadvertencies in design and erection work. A common response to this question would be that the plastic deformation capacity of the structural steel materials would cope with such unforeseeable disturbances. However, with increasing steel strength, the risk of brittle fracture increases and the relied-upon plastic deformation capacity may disappear. Another approach is to cope with such disturbances by lowering the level of stress that may be imposed on structural members without relying on their plastic deformation capacity. This might be effective especially in designing columns, which bear the whole weight of a building.

The new steel was developed not only to reduce the costs for fabrication and construction of building frames but also to promote the building design philosophy to use columns exclusively in the elastic region and thus to enhance the seismic resistance of buildings. It is expected that the developed steel, together with the building design philosophy of elastic columns, is widely applied, and its application spreads from high-rise buildings to middle- to low-rise buildings to enhance the reliability and safety of steel structure buildings.

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