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The body can be viewed from the next page.

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1 Introduction

Super HISLEND-H,[1-4] a range of H-shape steel with constant external dimensions which Kawasaki Steel introduced in November 1989, is a structural steel to be used for columns and beams of the steel-reinforced concrete (SRC) structure and steel (S) structure. Since its introduction, the range of sizes has been sequentially increased to satisfy customer demands, and since April 1992, 204 sizes have been manufactured. By exploiting its feature of higher dimensional accuracy than that of conventional rolled H-shapes, Super HISLEND-H has earned the reputation as a substitute for welded H-shapes.

In the following, this paper introduces the development of peripheral techniques for which Kawasaki Steel is now striving to expand the market of Super HISLEND-H for building steel structures, and steel reinforced concrete structures.

2 In-plane Elasto-Plastic Behavior Analysis of a Beam-Column

2.1 Object of the Analysis

In recent years, problems arose involving the defective quality of steel materials for construction purposes to a notable concern of the society, making the Construction Ministry open up the comprehensive technology development program entitled "the Development of New Materials and New Materials-Utilizing Technologies for Construction Projects." Meanwhile some new findings pointed out that the variation of yield point of steel materials not only affect final yield strength range of structural members, but also change the plastic hinge forming position of statistically indeterminate structures, or even collapse structure of steel frames conceived in designing stage.[5] As a matter of fact, a number of inquiries were sent in by structural designers about yield point variations and the range of yield ratio.

Against these backdrops, this paper has made some analytic study of plastic deformation capacity of H-shape steels based on the manufacturing results of Super HISLEND-H and assuming the variation of mechanical properties of steel materials.

2.2 Analysis Method

Model curves of the stress-strain relationship for the steel material are shown in **Fig. 1**. As the steel material, SM490 was assumed, and from the manufacturing records, three kinds of stress-strain relationship were assumed as cases 1, 2 and 3, whose yield points were the minimum, average and maximum, respectively. Of the other constants which define the stress-strain relationship, the hardening start strain (ε_{st}) and ultimate strain (ε_u) are shown by the following equation:

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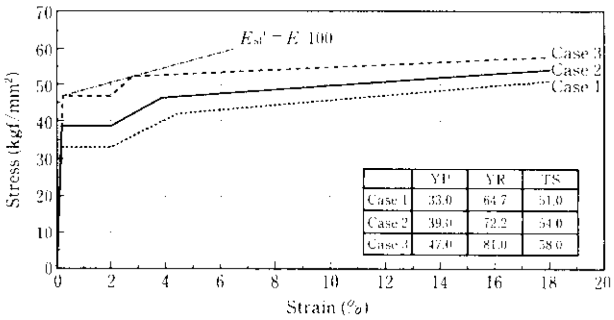


Fig. 1 Stress-strain curves for numerical analysis

Fig. 2 Loading condition

$$\varepsilon_{st} = 2(\%),\ \varepsilon_u = 18(\%) \cdots\cdots\cdots\cdots\cdots\cdots (1)$$

The strain hardening slope (E_{st}') was assumed to be 1/100 of Young's modulus ($E = 21\,000$ kgf/mm^2), and the stress level at a deflected point after hardening was assumed to be the median value between the yield point and tensile strength.

The cross section of the H-shape for analysis was set to 500 mm x 200 mm x 9 mm x 22 mm, and the slenderness ratio to 30. The loading conditions and boundary conditions are shown in **Fig. 2**. The axial compressive force ratio ($p = N/N_y$) was set to 0, 0.2, 0.4 and 0.6.

In the analysis, the moment-curvature relationship with respect to various axial compressive force ratios were first obtained. For this, it was assumed that plane preservation was valid, axial compressive force remained constant, and the bending moment gradually increased, while residual stress was ignored. Next, from this moment-curvature relationship, the moment-rotational angle relationship was obtained by the column deflection curve (CDC) method.

2.3 Analysis Results

The relationship between edge-region moment and rotational angle is shown in **Fig. 3**, and the ductility factor (η) obtained from this relationship is shown in **Fig. 4**. Ductility factor η was obtained by dividing the end rotational angle at the point of maximum strength by the elastic rotational angle at the point of full plastic strength, and then subtracting 1.0 from the quotient thus obtained.

The full plastic strength and maximum strength were both the maximum for case 3 in which the yield point and tensile strength were high. However, the ductility

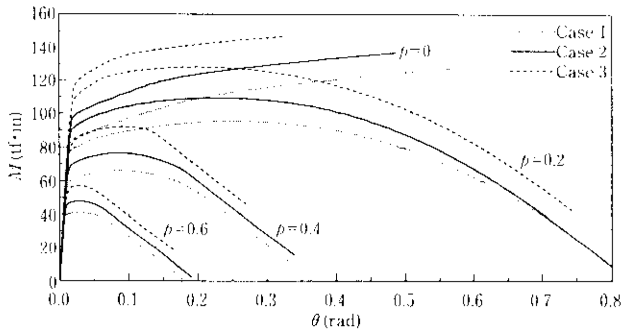


Fig. 3 M-θ curves (SM490)

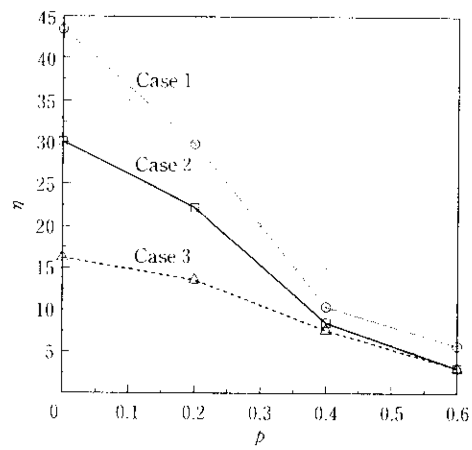


Fig. 4 Ductility of Super HISLEND H

was the minimum in case 3. It is well-known that, as the yield ratio increases, the ductility of a member decreases, and the present analysis shows a similar result.

The ductility necessary for a member has no definite index, and the results from the analysis are limited; hence, it is impossible to say that Super HISLEND-H has sufficient ductility simply from the present results. However in the case of an axial compressive force ratio of 0.4 or less which is practically used, the ductility factor of Super HISLEND-H reaches 8 or more and there would be no problem in the application of this H-shaped steel.

3 Dynamic Behavior of a Joint between Vertical Stiffener Type SRC Column and S Beam

3.1 Object of the Study

The reinforcement used for a beam-to-column joint in SRC which uses Super HISLEND-H is normally of the horizontal stiffening plate type. However, the horizontal stiffeners generate a filling problem with concrete, with work on the steel frames also made complicated. On the other hand, the vertical stiffening plate type, which was introduced in the "Standard for Structural Calculation of Steel-Reinforced Concrete Structures[6]" allows good filling with concrete and easy working. However, the dynamic bahavior of the vertical stiffening plate joint has much to be clarified as yet. Morita et al. have studied reinforement by a vertical stiffening plate for a steel beam, considering the case where the elements of a vertical stiffened column joint form an exact cruciform cross section, with the width of the column flange and that of the beam flange being equal.[7] The present paper reports the case where the width of the column flange and that of the beam flange are different, and particularly, the results of an experimental and analytical investigation into the strength evaluation of a vertical stiffener type beam-to-column joint when the column element is assumed to be a T-type cross section. This work is the result of a joint study between the authors and Professor Morita of Chiba University.[8]

3.2 Test Method

Morita et al.,[7] in carring out their experiments on specimens having the similar shape as series-B in **Fig. 5**, had the beam flange (${}_bb$) and column flange width (${}_cb$) the same, but changed the vertical stiffening plate width (${}_sh$), vertical stiffening plate thickness (${}_st$) and column flange thickness (${}_ct_f$). As a result, in a specimen column which was filled with concrete, the vertical stiffening plate developed tensile yielding, and even if the width of the vertical stiffening plate was increased beyond the specified width, no increase in the joint strength could be expected. It was also mentioned that the joint strength could be accurately evaluated Eq. (2) in Sec. 3.4.

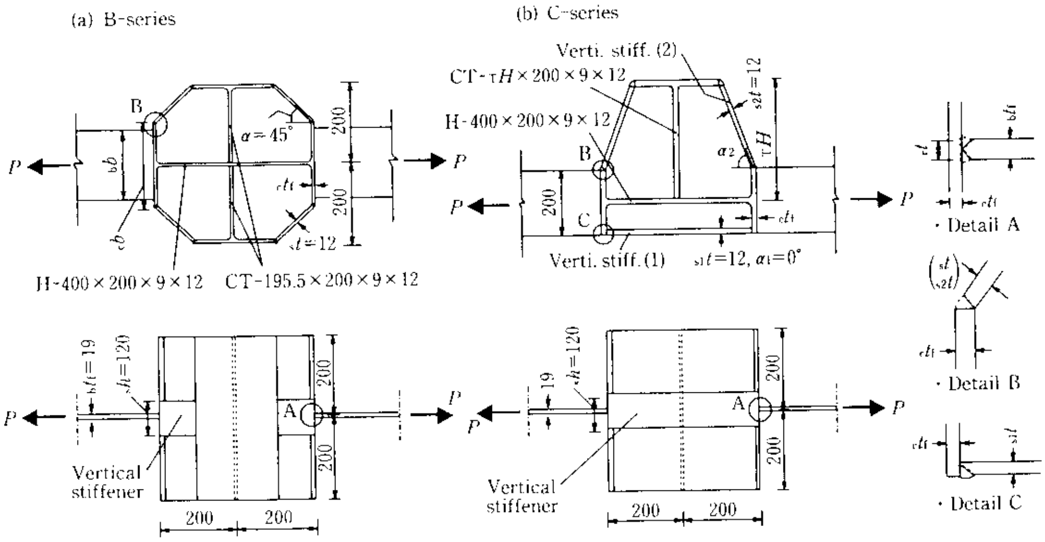
In the present test, therefore, a specimen was used in which width (${}_sh$) of the vertical stiffening plate was kept constant with respect specimen C-B1 at 120 mm, so as to satisfy the applicable range of Eq. (2). Then, in the case of series-B, the ratio between ${}_bb$ and ${}_cb$ was changed from 0.5 to 1.0, and the effect of beam width was investigated; in the case of series-T, the effect of fitting angle of the vertical stiffening plate was examined by changing height ${}_TH$ of the T-shape steel. For both series-B and T, four specimens each were used, their shapes being shown in Fig. 5.

The structural type of steel column used was one in which concrete was filled into the space bounded by the column flange and vertical stiffening plate. The material used steel shapes and plates of SM490A, and ordinary grade concrete. The mechanical properties of these materials are shown in **Table 1**.

Table 1 Mechanical properties of cruciform specimen

Material	YP (tf/cm²)	TS (tf/cm²)	El (%)	F_c*3 (kgf/cm²)
SM490 PL-9	4.36	5.38	24	
PL-12*1	3.71	5.21	26	
PL-12*2	3.62	5.17	25	
PL-19	3.95	5.38	26	
Concrete				187.97 161.13*4

*1 Column flange *3 Compressive strength
*2 Vertical stiffener *4 C-T3, C-T4



	C-B1	C-B2	C-B3	C-B4
${}_bb$(mm)	200	175	150	100

	C-T1	C-T2	C-T3	C-T4
${}_TH$(mm)	195.5	245.5	295.5	345.5
α_2(°)	45	56	63	68

Fig. 5 Geometry of cruciform specimens

The specimen was subjected to monotonous tensile loading, and the deformation and strain at various parts of the specimen under loading were measured.

3.3 Test Results

Load (P)-deformation (Δ) curves of series B and T specimens are shown in **Figs. 6** and 7, respectively. In these figures, the point indicated by "●" is the full plastic strength ($_eP_r$) of the joint obtained by the general yield point method. In the C-B4 specimen, the full plastic strength of the connection is greater than the yield strength of the beam flange, so that the full plastic strength of the connection cannot be evaluated; hence the full plastic strength was obtained from the curve (shown as a broken line) in which the deformation of the beam was excluded.

In series-B specimens, the strength of the connection decreases as the beam width is reduced, the strength of specimens C-B2, C-B3 and C-B4 being 0.96 times, 0.82 times and 0.67 times, respectively, that of C-B1. From the strain in various parts of the specimens, the yield sequence of various parts of the steel frame can be summarized as follows:

(1) C-B1: Tensile yielding of the column web → tensile yielding of the vertical stiffening plate → out-of-plane flexural yielding of the column flange → tensile yielding of the beam flange
(2) C-B2: Tensile yielding of the column flange → tensile flexural yielding of the vertical stiffening plate → out-of-plane flexural yielding of the column flange → tensile yielding of the beam flange
(3) C-B3: Tensile yielding of the column web → out-of-plane flexural yielding of the column flange → out-of-plane flexural yielding of the vertical stiffening plate → tensile yielding of the beam flange
(4) C-B4: Out-of-plane yielding of the column web → out-of-plane yielding of the column flange → tensile yielding of the beam flange → out-of-plane flexural yielding of the vertical stiffening plate

In this way, as the beam flange width is decreased, yielding of the vertical stiffening plate changes from tensile to out-of-plane flexural yielding.

Next, in the T-series specimens, the connection strength decreases as the height $_TH$ of T-shaped steel increases, the strength of C-T2, C-T3 and C-T4 being 0.96 times, 0.91 times and 0.89 times, respectively, that of C-T1.

The yield sequence in various parts is the same as that of C-B1 in series-T, but as $_TH$ increases, tensile yielding of the vertical stiffening plate tends to approach out-of-plane flexural yielding of the column flange.

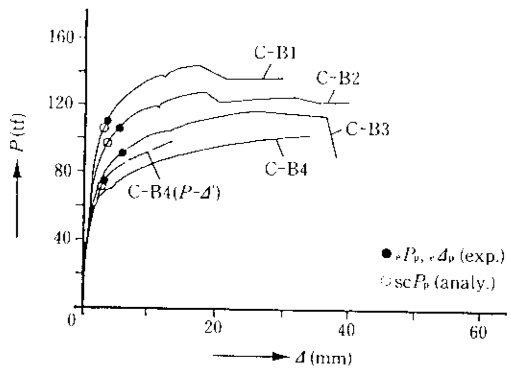


Fig. 6 Experimental P-Δ relationships of B-series cruciform specimens

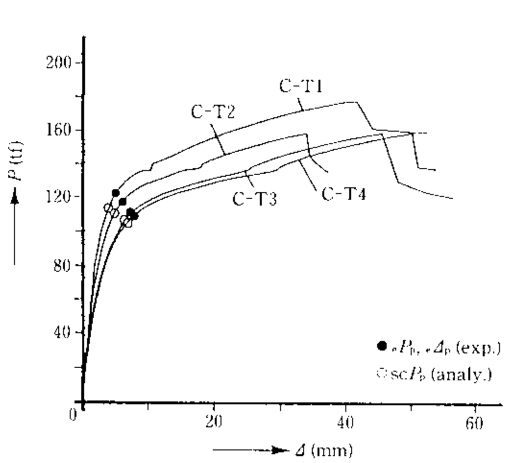


Fig. 7 Experimental P-Δ relationships of T-series cruciform specimens

3.4 Analysis of the Joint Strength

From the test results, it was found that, if the beam flange width was less than the width of the column flange, yielding of the vertical stiffening plate changed from tensile to out-of-plane flexural yielding. Therefore, the authors evaluated the connection strength by the yield line theory, assuming the following three types of collapse mechanism:

(1) Collapse Mechanism T

This mechanism was applied to tensile yielding of the vertical stiffening plate. In this case, the mechanism is the same as that shown by Morita et al.,[7] and the connection strength is given by the following equations:

$$\left.\begin{aligned}
{}_{sc}P_p &= 4\sqrt{{}_fM_p \cdot {}_cb(2{}_sT_y\cos\alpha + {}_wT_y)} \\
&\quad + (2{}_sT_y\cos\alpha + {}_wT_y)t \\
&\qquad \cdots\cdots {}_sh \geqq 4\sqrt{\frac{{}_fM_p \cdot {}_cb}{2{}_sT_y\cos\alpha + {}_wT_y}} + t \\
{}_{sc}P_p &= 4\sqrt{[{}_fM_p \cdot {}_cb - {}_sT_y\cos\alpha({}_sh - t)^2/8]{}_wT_y} \\
&\quad + 2{}_sT_y\cos\alpha \cdot {}_sh + {}_wT_y \cdot t \\
&\qquad \cdots\cdots {}_sh < 4\sqrt{\frac{{}_fM_p \cdot {}_cb}{2{}_sT_y\cos\alpha + {}_wT_y}} + t
\end{aligned}\right\} \quad (2)$$

where $t = {}_rt + {}_ct_f$

${}_fM_p$: Yielding moment per unit length of the column flange

${}_wT_y$: Tensile yielding strength per unit length of the column web

${}_sT_y$: Tensile yielding strength per unit length of the vertical stiffening plate

(2) Collapse Mechanism M

When yielding of the vertical stiffening plate is of the out-of-plane flexural type, the connection strength can be evaluated by the following equations if mechanism M shown in **Fig. 8** is assumed:

$$
\left.
\begin{aligned}
&{}_{sc}P_p = 4\sqrt{\frac{{}_fM_p\cdot{}_cb\{4({}_fM_p+{}_sM_p)+{}_wT_y\cdot m\}}{m}} + \left\{\frac{2({}_fM_p+{}_sM_p)+{}_wT_y\cdot m}{m}\right\}t \\
&\qquad\cdots\cdots {}_sh \geqq 4\sqrt{\frac{{}_fM_p\cdot{}_cb\cdot m}{4({}_fM_p+{}_sM_p)+{}_wT_y\cdot m}}+t \\
&{}_{sc}P_p = 4\sqrt{\frac{{}_fM_p\cdot{}_cb(4{}_fM_p+{}_wT_y\cdot m)}{m}} + \frac{2({}_fM_p\cdot t+{}_sM_p\cdot{}_sh)+{}_wT_y\cdot m\cdot t}{m} \\
&\qquad\cdots\cdots {}_sh < 4\sqrt{\frac{{}_fM_p\cdot{}_cb\cdot m}{4{}_fM_p+{}_wT_y\cdot m}}+t
\end{aligned}
\right\} \quad (3)
$$

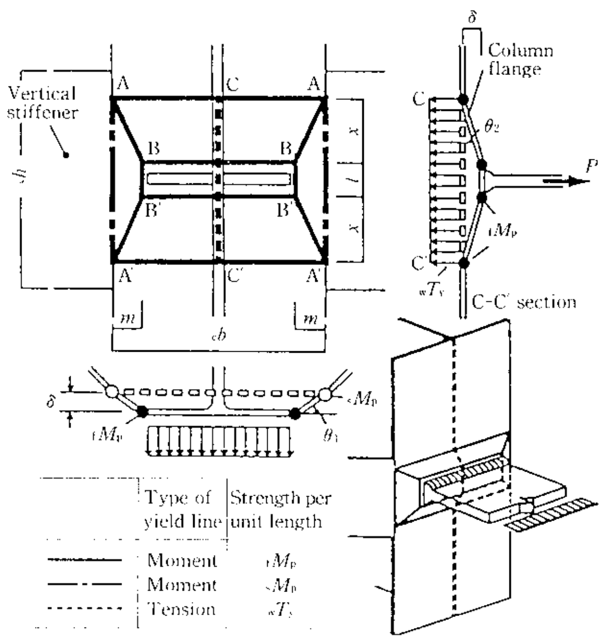


Fig. 8 Yield line mechanism M in steel-concrete column for B-series cruciform specimen

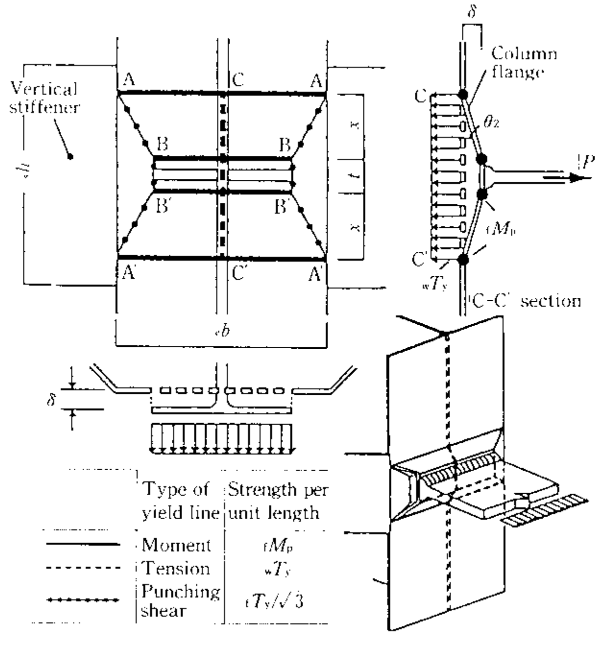


Fig. 9 Yield line mechanism S in steel-concrete column for B-series cruciform specimen

(3) Collapse Mechanism S

When yielding by punching shear occurs in the column flange, the strength of the connection can be evaluated by the following equations, if mechanism S shown in **Fig. 9** is assumed:

$$
\left.
\begin{aligned}
&{}_{sc}P_p = \frac{2{}_fM_p({}_cb+{}_bb)}{x} + 2\frac{{}_fT_y}{\sqrt{3}}\left\{\sqrt{x^2+\left(\frac{{}_cb-{}_bb}{3}\right)^2}+t\right\} + {}_wT_y(x+t) \\
&2\frac{{}_fT_y}{\sqrt{3}}x^3 + {}_wT_y\cdot x^2\sqrt{x^2+\left(\frac{{}_cb-{}_bb}{2}\right)^2} - 2{}_fM_p\cdot({}_cb+{}_cb)\sqrt{x^2+\left(\frac{{}_cb-{}_bb}{2}\right)^2} = 0
\end{aligned}
\right\} \quad (4)
$$

In the foregoing strength evaluation equations, it is considered that the minimum values are the strength values in the case of the series-B specimens.

In the case of the series-T specimens, the width of the column flange is equal to that of the beam flange; hence, mechanisms M and S shown for the B-series specimens are not applicable. However, the test results show that the strain in the vertical stiffening plate near the beam flange reaches, at both the front and back, the tensile yielding strain; hence, an analysis can be made by using mechanism T as shown for the series-B specimens. The results of this analysis are shown below.

$$
{}_{sc}P_p = 4\sqrt{{}_fM_p \cdot {}_cb({}_{s1}T_y \cos\alpha_1 + {}_{s2}T_y \cos\alpha_2 + {}_wT_y)} + ({}_{s1}T_y \cos\alpha_1 + {}_{s2}T_y \cos\alpha_2 + {}_wT_y)t \quad \cdots\cdots {}_sh \geqq 4\sqrt{\frac{{}_fM_p \cdot {}_cb}{{}_{s1}T_y \cos\alpha_1 + {}_{s2}T_y \cos\alpha_2 + {}_wT_y}} + t
$$

$$
{}_{sc}P_p = 4\sqrt{\left[{}_fM_p \cdot {}_cb - \frac{({}_{s1}T_y \cos\alpha_1 + {}_{s2}T_y \cos\alpha_2)({}_sh - t)^2}{16}\right]{}_wT_y} + ({}_{s1}T_y \cos\alpha_1 + {}_{s2}T_y \cos\alpha_2){}_sh + {}_wT_y \cdot t \quad \cdots\cdots {}_sh < 4\sqrt{\frac{{}_fM_p \cdot {}_cb}{{}_{s1}T_y \cos\alpha_1 + {}_{s2}T_y \cos\alpha_2 + {}_wT_y}} + t
$$

$$\cdots\cdots(5)$$

where ${}_{s1}T_y$: Tensile yield strength per unit length of the vertical stiffening plate (1)

${}_{s2}T_y$: Tensile yield strength per unit length of the vertical stiffening plate (2)

Test and analytical values for the fully plastic strength of the B-series and T-series specimens are summarized in **Table 2**. In the aforementioned P-Δ curves (Figs. 6 and 7), analytical values are shown by the "●" mark.

These tables indicate that the analytical values have accurately reproduced the test values, and confirm the validity of these equations for evaluating the connection strength.

4 Conclusions

The in-plane elasto-plasto behavior analysis of Super HISLEND-H was reported by taking into consideration the variations of its mechanical properties. Further, as an example of peripheral technology development of SRC structures, an experiment was reported on the experiment on the joints between vertical stiffener type SRC-column and S type beam.

As a result, it was found that Super HISLEND-H has sufficient ductility within the range of the variation of its mechanical properties. Regarding the joint between vertical stiffener type SRC column and S-type beam, its effectiveness was confirmed, with a finding of ductility evaluation equations, making it possible for an application to actual structures.

Table 2 General yield strength of B-series and T-series specimens

Specimen	Experimental ${}_eP_p$ (tf)	Analytical ${}_{sc}P_p$ (tf)	${}_eP_p/{}_{sc}P_p$
C-B1	111.3	106.8	1.04
C-B2	106.7	97.0	1.10
C-B3	91.3	91.3	1.00
C-B4	74.2	72.7	1.02
C-T1	123.7	114.8	1.08
C-T2	118.7	112.0	1.06
C-T3	112.0	108.3	1.03
C-T4	110.7	106.8	1.04

5 Acknowledgement

Studies on the vertical stiffening plate type of beam-to-column joint were carried out jointly with Professor Koji Morita of Architectural Department, Faculty of Engineering of Chiba University. The authors express their deep appreciation to Professor Morita for his co-operation, and also to the members of the Morita Research Team for their valuable assistance rendered during the testing.

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