SEISMIC PERFORMANCE OF NEW TYPE STEEL-CONCRETE COMPOSITE STRUCTURES CONSIDERING CHARACTERISTIC BOTH SRC AND CFT STRUCTURES

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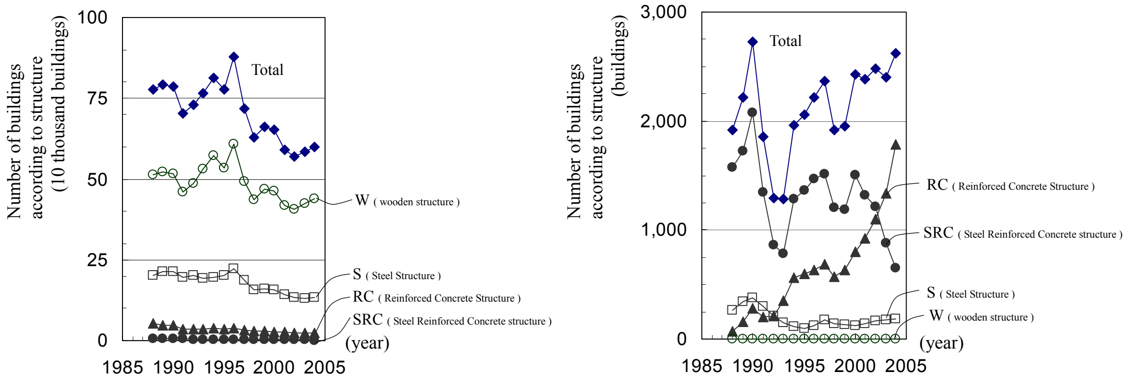
ABSTRACT :

From the situation of a recent construction cost, the demand for ordinarily SRC structures decreases. This factor is that the excellent mechanical property of SRC structures cannot be demonstrated. It is because the steel is used, it is higher than that of RC structures on both sides of the material and construction. Then, it is researching the steel-concrete composite structures of new systems as a structure to have the earthquake-resistant, the workability, and the economy. This new type steel-concrete structure with advantage of RC structures is the best use of the characteristic of SRC structures and CFT structures.
To obtain the seismic performance of new system of composite structure, 6 columns specimens (Hereafter, it is called B series) were tested. Steel-concrete composite columns have "diagonal steel section" and "steel plate boxes shape section (Hereafter, it is called the box shape steel pipe) had only to the top and the bottom of columns".
New type composite columns with diagonal section have about twice the bending yield strength with ordinarily section. And, it has the deformation capacity. It obtained of the basic characteristic of new type steel-concrete composite structures, and the evaluation of ultimate strength and ductility of steel-concrete columns is examined.
In conclusion, steel-concrete composite structures of proposed new type have been understood that it has seismic performance and economical construction be done compared with ordinarily SRC structures and CFT structures.

KEYWORDS: New Type Steel-Concrete Composite Structures, Concrete Filled Tube Structures, Steel Reinforced Concrete Structures, Diagonal Steel Section, Seismic Performance

1. INTRODUCTION

According to the building trend statistics investigation [1] that Japanese Ministry of Land, Infrastructure, Transport and Tourism carries out every year, the construction circumstances of the building in Japan change. About the number of buildings which is commencement of work, the total of a change is shown in Figure-1(a), the building of 10-15 floors of a change is shown in Figure-1(b).The number of buildings show here regards count of from January of 2005 to December of 2005 as a plan in 2005.
It pays attention to the transition of the structural model adopted in the building of 10-15 floors. SRC structures decreases rapidly. On the other hand, the RC structure is rapidly adopted.
As for the RC structure, a proper structural design and the structure design became possible by revision of building standard law in 2000, reinforcement and concrete are strengthening, and introduction of technology of base-isolation and seismic control. The SRC structures used the steel, it is higher than RC structures on both sides of the material and construction. Therefore, it is a situation in which the excellent mechanical property that an ordinarily SRC structures has cannot be reflected.
Then, we are researching the steel-concrete composite columns of new systems as a structure to have the earthquake-resistant, the workability, and the economy. This new type steel-concrete structure which can be used by changing into RC structure is the best use of the characteristic of SRC columns and CFT columns.



(a) Total (b) The building of 10-15 floors

Figure-1 Transition of number of buildings of commencement of work according to structure

2. PLANNING OF EXPERIMENT

2.1. Outline of Experiment

The specimens planned six columns shown in Table-1. The main axis of the steel frame was rotated by 45 degrees and to demonstrate almost twice the ultimate flexural strength compared with a ordinarily cross type steel frame section. The section where this main axis was inclined 45 degrees is called X section. The ratio of steel section on planned specimens shown in Table-2. This X section improves the ultimate flexural strength of the SRC columns by the same amount of amount of the single H section steel. In addition, internal concrete is restrained by installing the stiffening plate in the top and the bottom like the ring (Hereafter, it is called the reinforcement steel band). The reason for it is that the deformation capacity is increased compared with the high axial stress. The reinforcement steel band was installed on the X section type steel frame by the fillet welding. And, the hole is made for the web steel part to achieve the character on the whole, and the specimens which improves concrete adhesion power is planned additionally about concrete (Photo-1).

In a common factor, specified concrete strength is Fc30 and the column inside measurement length is 900mm. The variable factors are axial compression ratio, installation of reinforcement steel band is be or not be and angle of built-in cross type steel section. The section of specimen and the sectional form are shown in Figure-2 and Figure-3.

Table-1 Test Program

Test Speci -mens	Sectional Form (mm)	Axial Com -pression Ratio ${}_{sc}n$	Steel Skeleton			Steel of Ratio ${}_sp$ (%)	Reinforcement Steel Band	
			Section	Angle	Hole		Thickness t_s (mm)	Length l_s (mm)
B-1	300 ×300	0.3	2H-300x150x6.5x9 (Cross section)	0°	—	7.33	—	—
B-2					40φ-@150			
B-3			2H-350x75x7x11 (X section)	45°	—	13.22		
B-4					40φ-@150			
B-5							6	150
B-6		0.6						

Note 1) Axial Compression Ratio ${}_{sc}n = N/(b\cdot D\cdot \sigma_B + {}_sA\cdot{}_s\sigma_y) = {}_cn/(1+{}_s\phi)$
here, ${}_s\phi = ({}_sp\cdot{}_s\sigma_y)/\sigma_B$

Note 2) The angle of the steel section is assumed to be angle (θ) to the direction of the main axis.

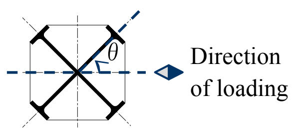
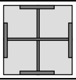
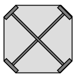
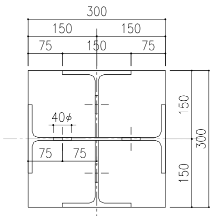


Table-2 Section

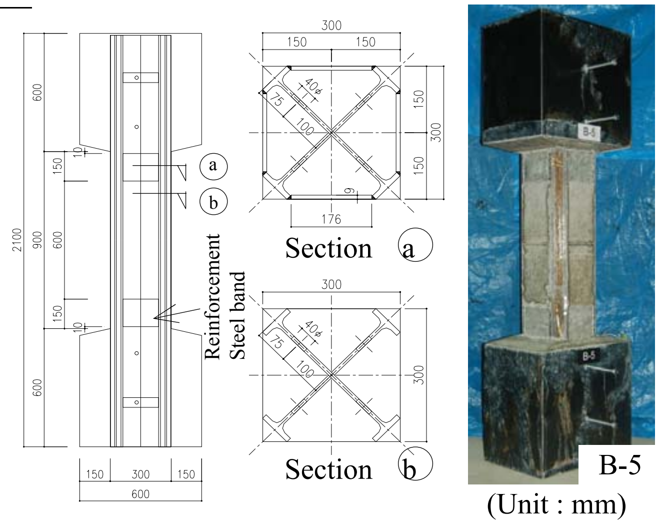
Test Specimens		The ratio of flange steel $_{s}p_{f}$' (%)	The ratio of web steel $_{s}p_{w}$' (%)	The ratio of steel $_{s}p$ (%)
B-1		6.00	4.33	10.33
B-2		[1.00]	[1.00]	[1.00]
B-3				
B-4		3.91	5.87	9.72
B-5		[0.65]	[1.34]	[0.94]
B-6				

Note 1) $_{s}p_{f}' = {}_{s}A_{f} / {}_{c}A'$, $_{s}p_{w}' = {}_{s}A_{w} / {}_{c}A'$
$_{s}p = {}_{s}A / {}_{c}A'$
here $_{s}A_{f}$: area of steel flange
$_{s}A_{w}$: area of steel web
$_{s}A$: area of steel portion
$_{c}A'$: area of concrete portion
Note 2) [ ] numerical value shows the ratio of X section steel when the cross section steel is assumed to be 1.00.

Photo-1 Situation of perforated steel web frame



(Unit : mm)
Figure-2 Sectional form [B-1,B-2]



(Unit : mm)
Figure-3 Section of Specimen and the Sectional Form [B-5,B-6]

2.2. *Outline of Experiment*

The loading used "Seismic Loading Device" of the Fukuyama University in Hiroshima, Japan. The loading cycles are repeatedly of the positive and negative. The experiment is angle of rotation of member R (= δ_{H}/H ; δ_{H} : The horizontal displacement, H : the column inside measurement span) by displacement control in the column top. The experiment ends for when the ultimate strength decreases to 70% of the maximum strength or when the axis strength maintenance becomes impossible.

2.3. *Material Characteristic*

Material strength used to evaluate ultimate stress uses the result of the material testing. Concrete compressive. strength used to be σ_{B}=44.8N/mm^{2} of the value before it experimented and after it had experimented. And the material characteristic of steel is shown in Table-3

Table-3 Material Characteristic of Steel

Steel Section	Thickness		Yield Stress σ_y (N/mm^2)	Tensile Stress σ_u (N/mm^2)	Yield ratio $\frac{\sigma_y}{\sigma_u}$	Elongation (%)
H-300x150x6.5x9 [SS400]	t6.5	Web	353.3	453.7	0.78	22.1
	t9	flange	306.7	488.7	0.63	25.5
H-350x75x7x11 [SS400]	t7	Web	348.3	491.0	0.71	23.2
	t11	flange	321.7	385.7	0.83	20.1
Reinforcement Steel Band	t6	—	298.3	407.4	0.73	21.6

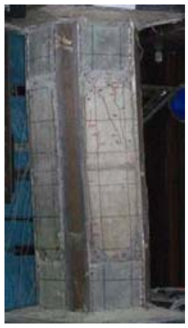
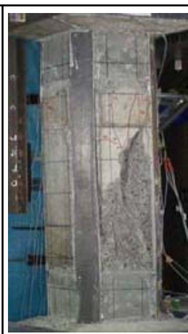
3. Experiment Result

3.1. Failure State

Figure-4 shows the final failure state.

On maximum strength specimens of non reinforcement steel band have cracks of flexural occurs on top and bottom of column. On maximum strength specimens of reinforcement steel band have not cracks of flexural occurs on top and bottom of column. The local buckling was not caused in all specimens.

The buckling of the steel was not generated in the specimen with the reinforcement steel band of high axial load (${}_{SC}n$=0.6). The specimen of high axial load (${}_{SC}n$=0.6) by the same shape had the concrete cracks and crashs than the specimen of low axial load (${}_{SC}n$=0.3). However, it had the ability of the maintenance of the axial tension in displacement angle of rotation of member R=2.0×10^{-2}rad.

	B-1	B-2	B-3	B-4	B-5	B-6
${}_{sc}n$	${}_{sc}n$=0.3					${}_{sc}n$=0.6
	non reinforcement steel band				reinforcement steel band ${}_{s}t$=6	
						
Q_{max}	676kN	654kN	760kN	721kN	788kN	704kN
R_u	5.0×10^{-2}rad	5.0×10^{-2}rad	5.0×10^{-2}rad	4.5×10^{-2}rad	5.0×10^{-2}rad	2.0×10^{-2}rad

Note) Q_{max} : maximum strength (kN) , R_u : ultimate angle of rotation of member (10^{-2}rad)

Figure-4 Final Failure State

3.2. Hysteresis Characteristic

Figure-5 shows the hysteresis characteristic. In the load-deformation curve, the horizontal load Q(kN) is shown in the ordinate and displacement angle of rotation of member R(10^{-2}rad) is shown in the abscissa. Here, the axial strain becomes 1% when axial displacement δ_N is 9mm.

The specimen which received the high axial load had the deformation capability to displacement angle of rotation of member R=1.5×10-2rad (If ${}_{SC}n$=0.6 is replaced, it becomes ${}_{C}n$=1.0, Reference: Table-1). Therefore, the reinforcement steel band functioned effectively as prevention crushing of concrete.

4. Calculation of Ultimate Strength

4.1. Ultimate Strength of Steel-Concrete Columns

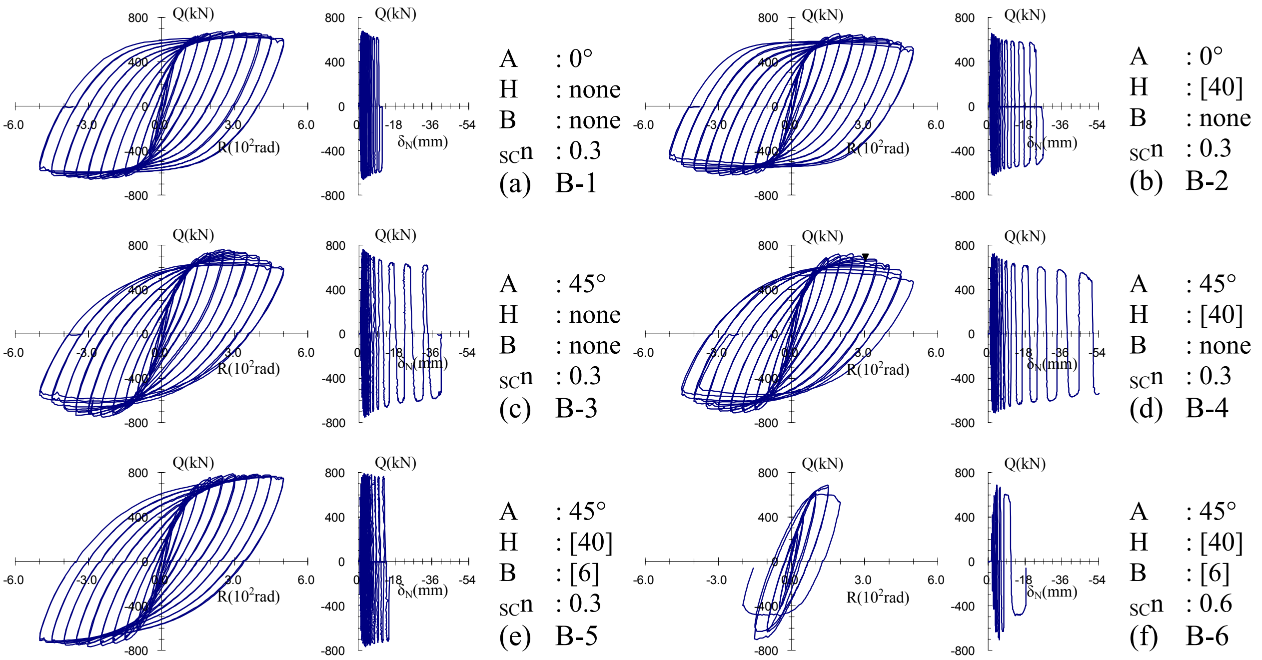
In the SRC member, as for the steel part and the concrete part, each bond strength is extremely small. On this account the steel part and the concrete part resist bending stress and shearing stress separately. The SC member becomes similar to the SRC member. Thus, ultimate strength of a member is given by Eq. 4.1 [2],[3].

$${}_{SC}Q_U = {}_{C}Q_U + {}_{S}Q_U \tag{4.1}$$

Here ${}_{SC}Q_U$: ultimate shear strength of member

${}_{C}Q_U$: ultimate shear strength of concrete portion

${}_{S}Q_U$: ultimate shear strength of steel portion



A : Angle of the Steel Section From the Main Axis.(0,45°), H : Hole of web steel [Diameter (mm)]
B : Reinforcement Steel Band [Thickness (mm)], ${}_{SC}n$: Axial Compression Ratio

Figure-5 Hysteresis Characteristic

It is requested in consideration of the ultimate strength, "${}_{C}Q_U$" and "${}_{S}Q_U$" shall be as given by Eqs. 4.2 and 4.3.

$$ {}_{C}Q_U = \min({}_{C}Q_{sU}, {}_{C}Q_{bU}) \tag{4.2} $$

$$ {}_{S}Q_U = \min({}_{S}Q_{sU}, {}_{S}Q_{bU}) \tag{4.3} $$

Here ${}_{C}Q_{sU}$: ultimate strength in ${}_{C}Q_U$ based on shear failure of concrete portion
${}_{C}Q_{bU}$: ultimate strength in ${}_{C}Q_U$ based on flexural failure of concrete portion
${}_{S}Q_{bU}$: ultimate strength in ${}_{S}Q_U$ based on shear failure of steel portion
${}_{S}Q_{sU}$: ultimate strength in ${}_{S}Q_U$ based on flexural failure of steel portion

4.2. Ultimate Flexural Strength of members

It calculates according to the generalized superposed strength [4]. The ultimate flexural of cross section superposed strength of the concrete, strong axis of steel and weak axis of steel. The ultimate flexural of X section superposed strength of the concrete, flange of steel and web of steel. The X section is substituted for the section shown in Figure-6. As for the sectional area of the steel frame after it substitutes it as shown in Figure-7, the flange area is ${}_{f}t'\cdot D_1$ and the web area is ${}_{w}t'\cdot Ds'$. Ultimate flexural strength is given by Eq. 4.4.

$$ {}_{SC}Q_{bU} = 2\cdot{}_{SC}M_U/L \tag{4.4} $$

Here ${}_{SC}M_U = {}_{SC}m_U\cdot(b\cdot D^2\cdot Fc)$, ${}_{SC}N_U = {}_{SC}n_U\cdot(b\cdot D\cdot Fc)$

${}_{SC}n_i$ and ${}_{SC}m_i$ is given by Eqs. 4.5 through 4.7.

When ${}_{SC}n_2 \le {}_{SC}n_u \le {}_{SC}n_3$

$$ {}_{SC}m_u = \sqrt{2}\cdot d_{s1}\left({}_{s}\mu_f + \frac{1}{4}{}_{s}\mu_w\right) + \frac{\alpha_c}{2}\{{}_{SC}n_u + {}_{s}\mu_w\}\cdot\left(1-\frac{1}{\alpha_c^2\cdot k_3}\{{}_{SC}n_u + {}_{s}\mu_w\}\right) \tag{4.5} $$

When ${}_{SC}n_3 \le {}_{SC}n_u \le {}_{SC}n_4$

$$ {}_{SC}m_u = \frac{\alpha_c^3\cdot k_3}{8}\cdot\sqrt{2}\cdot d_{s1}\left({}_{s}\mu_f + \frac{1}{4} + {}_{s}\mu_w\right) \tag{4.6} $$

When ${}_{SC}n_4 \le {}_{SC}n_u \le {}_{SC}n_5$

$$ {}_{SC}m_u = \sqrt{2}\cdot d_{s1}\left({}_{s}\mu_f + \frac{1}{4}{}_{s}\mu_w\right) + \frac{\alpha_c}{2}\{{}_{SC}n_u - {}_{s}\mu_w\}\cdot\left(1-\frac{1}{\alpha_c^2\cdot k_3}\{{}_{SC}n_u - {}_{s}\mu_w\}\right) \tag{4.7} $$

Here ${}_{SC}n_2 = -{}_{s}\mu_w + \frac{\sqrt{2}}{4}\alpha_c\cdot k_3\left(\sqrt{2}\alpha_c - d_{s1}\right)$, ${}_{SC}n_3 = -{}_{s}\mu_w + \frac{\alpha_c^2}{2}\cdot k_3$

$$ {}_{SC}n_4 = {}_{s}\mu_w + \frac{\alpha_c^{\,2}}{2}\cdot k_3 \qquad , \quad {}_{SC}n_5 = {}_{s}\mu_w + \frac{\sqrt{2}}{4}\alpha_c \cdot k_3\left(\sqrt{2}\alpha_c + d_{s1}\right) $$

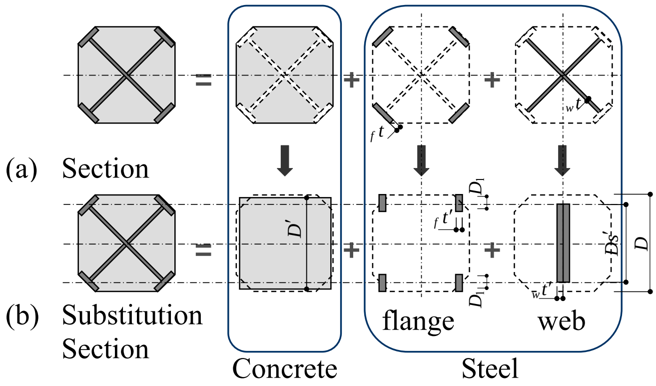


Figure-6 Section for Ultimate Flexural Strength (X Section)

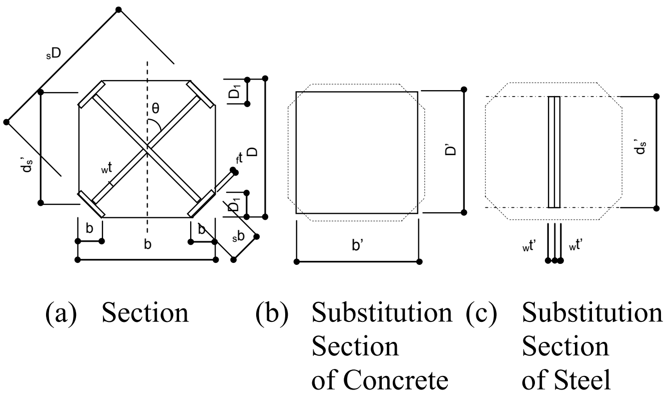


Figure-7 Notation for the Symbols used in Substitution Section

4.3. Ultimate Shear Strength of members

The section for the ultimate shear ultimate strength calculation is shown in Figure-8 through 10. Shear resistant at concrete portion is assumed for the arch action. As for concrete neutral axis, the compression region was adjusted to the position which became equal with the sectional area of the tensile region. The arch action is formed between force to concentrate on the center of gravity position of each section, and the angle of this arch action is assumed to be θ_i. It is given by Eqs. 4.8 through 4.11.

1) When the angle of steel section the main axis is 0 degrees (cross section)

$$ {}_{C}Q_{sU} = 2\cdot\nu_0\cdot\sigma_B\cdot A_0\cdot\tan\theta_0 \tag{4.8} $$

$$ {}_{S}Q_{sU} = {}_{w}t\cdot d_s\cdot{}_{w}\sigma_y\big/\sqrt{3} \tag{4.9} $$

Here $\quad A_0 = \dfrac{b\cdot D}{4}, \quad \tan\theta_0 = \sqrt{\eta_0^{\,2}+1} - \eta_0, \quad \eta_0 = \dfrac{2L}{D}$

2) When the angle of steel section the main axis is 45 degrees (X section)

$$ cQsU = 2\cdot\nu\cdot\sigma_B\cdot A_1\cdot\tan\theta_1\cdot\sqrt{\tan^2\theta_1+1} + 2\cdot\nu\cdot\sigma_B\cdot A_2\cdot\tan\theta_2\cdot\sqrt{\tan^2\theta_2+1} \tag{4.10} $$

$$ {}_{S}Q_{sU} = 2\cdot{}_{w}t\cdot d_s\cdot{}_{w}\sigma_y\big/\sqrt{3} \tag{4.11} $$

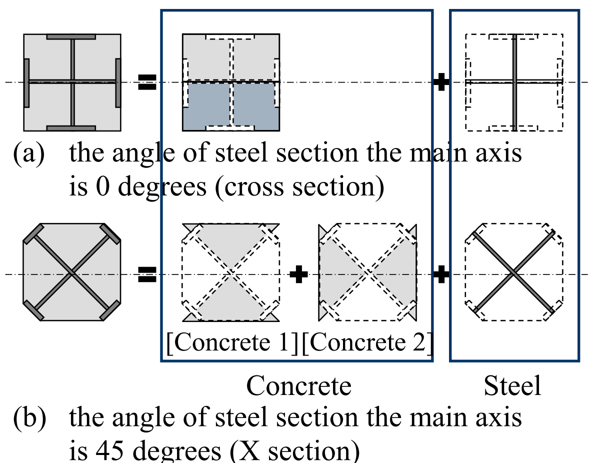


Figure-8 Section for Ultimate Shear Strength

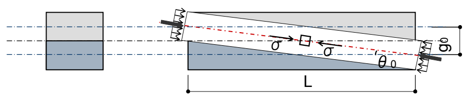


Figure-9 Concrete Shearing Resistance Mechanism in Cross Section (Arch Action)

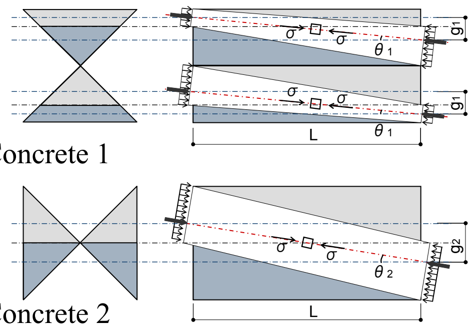


Figure-10 Concrete Shearing Resistance Mechanism in X Section (Arch Action)

$$\text{[Concrete 1]}\quad A_1 = \frac{b\cdot D}{8},\quad \tan\theta_1 = \sqrt{\frac{1}{\beta_1^{2}}\cdot\eta_1^{2}+\frac{1}{\beta_1}}-\frac{1}{\beta_1}\cdot\eta_1,\quad \eta_1=\frac{L}{2\cdot g_1},\quad g_1=\frac{3\sqrt{2}-2}{12}\cdot D,\quad \beta_1=\frac{4\sqrt{2}-3}{3-\sqrt{2}}(=1.68)$$

$$\text{[Concrete 2]}\quad A_2 = \frac{b\cdot D}{4},\quad \tan\theta_2 = \sqrt{\frac{1}{\beta_2^{2}}\cdot\eta_2^{2}+\frac{1}{\beta_2}}-\frac{1}{\beta_2}\cdot\eta_2,\quad \eta_2=\frac{L}{2\cdot g_2},\quad g_2=\frac{1}{3}\cdot D\quad ,\beta_2=2.0$$

4.4. Comparison between ultimate theoretical strength and experimental strength

The comparison between ultimate theoretical strength calculated by the method of the above-mentioned bearing capacity evaluation and the experimental strength is shown in Figure-11 and the Q-N correlation diagram is shown in Figure-12.

It can be said $Q_{exp}/{}_{SC}Q_U$ of specimen (B-1, B-3 ; it have not reinforcement steel band, and hole of web steel) is about 1.20, and it is almost appreciable. On the other hand, $Q_{exp}/{}_{SC}Q_U$ of the specimen which have holes in the steel web is large, $Q_{exp}/{}_{SC}Q_U$ of the specimen which have the reinforcement steel band is small.

As for this reason, the following three are thought.

1) Ultimate strength of the specimens with the steel web by installing the hole decreases.

2) Ultimate strength of the specimens with the reinforcement steel band is increased by confined effects of concrete.

3) Ultimate strength of the specimens is increased by buckling prevention of steel

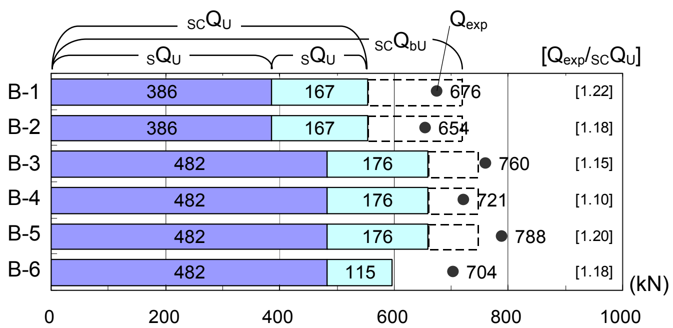
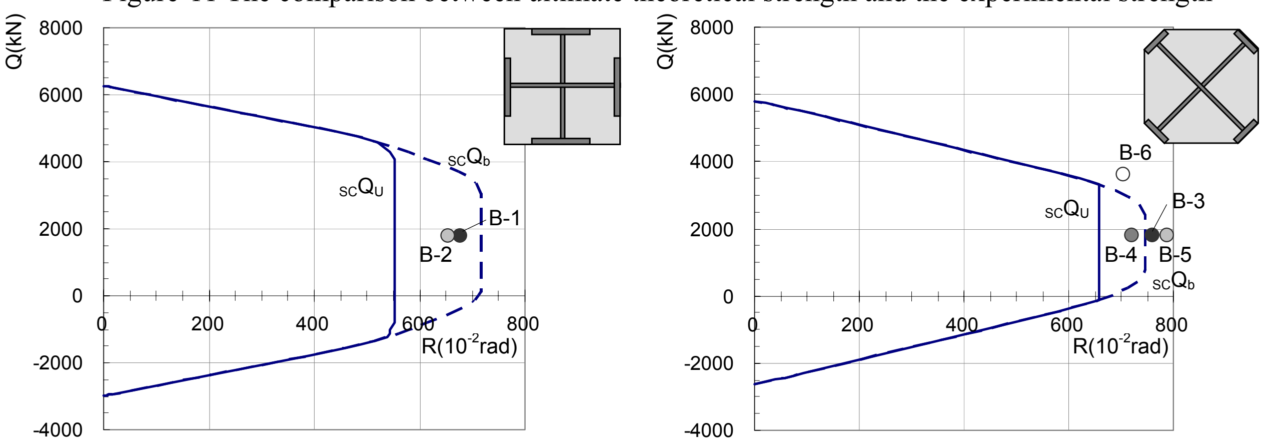


Figure-11 The comparison between ultimate theoretical strength and the experimental strength



(a) the angle of steel section the main axis is 0 degrees (cross section)

(b) the angle of steel section the main axis is 45 degrees (X section)

Figure-12 Q-N correlation diagram

5. Summary

Based on the observation of tests and analysis of data, the following conclusions were obtained.

1) The specimens with the X section type steel frame, it increases the maximum strength and horizontal displacement (deformation angle) of the maximum strength.

2) The steel frame and concrete adhesion power has improved to the specimen which made the hole for the steel web (perfobond strip). And the reinforcement steel band suppresses the buckling of the steel frame.

3) The deformation capacity improves by the reinforcement steel band under the high axial compressive stress. This reason is for the reinforcement steel band to prevent the crushing of concrete and the buckling of steel frame.
4) It was shown to have a performance excellent when the main axis of the cross type steel frame was inclined 45 times (X section).
5) Local buckling of the steel frame comes to cause it easily by losing a concrete thickness of the cover. However, the generation of local buckling of the steel frame is from now on at maximum strength, and a rapid yield strength decrease is not caused after local buckling is caused.

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Appendix

The list of the sign used by the expression is shown below.

θ : angle to main axis of steel frame
b : width of member , D : depth of member
b' : width of member after it substitutes it $(=\alpha_c \cdot b)$
D' : depth of member after it substitutes it $(=\alpha_c \cdot D)$
${}_sb$: width of steel , ${}_sD$: depth of steel
${}_sD'$: depth of steel after it substitutes it $(=\alpha_c \cdot D)$
${}_ft$: thickness of steel flange
${}_wt$: web thickness of full-web steel
${}_ft'$: thickness of steel flange after it substitutes it $(=\alpha_f \cdot {}_ft)$
${}_wt'$: web thickness of full-web steel after it substitutes it $(=\alpha_w \cdot {}_wt)$
${}_sA_f$: area of steel flange $(={}_ft \cdot {}_sb)$
${}_sA_w$: area of steel web $(={}_wt \cdot d_s)$
${}_sp_f$: flange steel ratio $(={}_sA_f/(b \cdot D))$
${}_sp_w$: web steel ratio $(={}_sA_w/(b \cdot D))$
${}_sp_c$: compression steel ratio $(={}_sA_c/(b \cdot D))$
${}_s\mu_f$: coefficient multiplied to compressive strength of flange steel portion determining limit to axial force in column $(={}_sp_f \cdot {}_f\sigma_y/Fc)$
${}_s\mu_w$: coefficient multiplied to compressive strength of web steel portion determining limit to axial force in column $(={}_sp_w \cdot {}_w\sigma_y/Fc)$

α_c : $\left(\alpha_c = \sqrt{(bD - 2b_1 \cdot D_1)/bD}\right)$
α_c : $\left(\alpha_c = \sqrt{(bD - 2b_1 \cdot D_1)/bD}\right)$
α_w : $(\alpha_w = d_s/d_s')$
when $\theta = 45°\ (=\pi/4\ rad)$, ${}_sD' = {}_sD/\sqrt{2}$, $\alpha_w = \sqrt{2}$
α_f : $(\alpha_f = {}_sb/D_1)$
when $\theta = 45°\ (=\pi/4\ rad)$, $D_1 = {}_sb/\sqrt{2}$, $\alpha_f = \sqrt{2}$
α_d : $(\alpha_d = d_s'/d_s)$
when $\theta = 45°\ (=\pi/4\ rad)$, $d_s' = d_s/\sqrt{2}$, $\alpha_d = 1/\sqrt{2}$
d_{s1} : $(=d_s/D)$, d_{s1}' : $(=d_s'/D = \alpha_d \cdot d_{s1})$
d_s : center distance between tension and compression chords of flanges of steel (when $\theta = 0°$)
d_s' : center distance between tension and compression chords of flanges of steel (when $\theta = 45°$) $(=\alpha_d \cdot {}_sd)$
Fc : design standard strength of concrete
Fc' : allowable compressive stress used in calculation for concrete portion of column $Fc' = k_3 \cdot Fc$
k_3 : $k_3 = 0.85 - 2.5{}_sp_c$
${}_f\sigma_y$: yield stress of flange steel
${}_w\sigma_y$: yield stress of web steel

A_i : area of compression zone
g_i : distance from center axis in compression zone at one end to center axis in compression zone of other end
θ_i : angle of arch action

σ_B : compressive stress
ν : factor of safety for buckling of concrete portion where $\nu = 1.0$