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MECHANICAL CHARACTERISTIC OF STEEL REINFORCED CONCRETE COLUMNS WITH LOW-STRENGTH CONCRETE

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Abstract

Steel reinforced concrete structure possesses the properties of both concrete and steel, and by appropriate design it is possible to provide good earthquake resistance in such structures. High-rise buildings of steel reinforced concrete construction showed good earthquake-resistant capacity under the Kanto earthquake (1923) as compared with ordinary reinforced concrete structures. Since then the encased structural system, a form of composite construction, has been employed in Japan for most building frames higher than seven stories. Though no steel reinforced concrete buildings had been collapsed by the previous earthquakes in Japan, it was reported that steel reinforced concrete buildings were damaged seriously owing to Hyogoken-Nanbu earthquake (1995) in Japan. Those buildings were existing buildings which had been built before the law revision in 1981.

After the Hyogoken-Nanbu earthquake, seismic evaluation and retrofit in concrete buildings were executed aggressively. According to some reports of seismic evaluations of existing reinforced concrete buildings, the presence of reinforced concrete buildings with low-strength concrete has been confirmed. In the standard for seismic evaluation of existing concrete buildings by the Japan Building Disaster Prevention Association (JBDPA), coverage of concrete compression strength is assumed to be 13.5N/mm^2 or more. It is necessary for evaluation and retrofitting to clarify the mechanical characteristics of low strength concrete and seismic performance of low strength concrete members and frames. Over the past few years, several studies have been made on mechanical characteristic of reinforced concrete members and frames with low-strength concrete. However, seismic evaluation and retrofit of steel reinforced concrete buildings with low-strength concrete has never been studied so far. Therefore, little is known about mechanical characteristic of steel reinforced concrete buildings with low-strength concrete.

In this research, shear strength of steel reinforced concrete columns with low-strength concrete are investigated through the seismic loading tests under the constant vertical load. The considered parameters in the tests are as follows, steel type, axial load level and concrete strength. Concrete specified design was assumed to be 9N/mm^2 and 18N/mm^2. Main discussion is concentrated on the maximum strength, the behavior before and after the attainment of the maximum strength and hysteretic characteristics. From the test results, it is found that the ultimate shear strength cannot be evaluated using the present equation recommended in JBDPA, design standard for the steel reinforced concrete columns with low-strength concrete. On the other hand, we confirmed that the ultimate shear strength can be estimated the equation based on the theory of truss and arch mechanism in Architectural Institute of Japan (AIJ) design for the steel reinforced concrete standard with low-strength concrete.

Keywords: seismic evaluation; *low-strength concrete*; *ultimate shear strength*

1. Introduction

The Japanese government enacted “the Act on Promotion of the Earthquake-proof Retrofit of Buildings,” in 1995, for the public buildings, such as schools, for seismic evaluation and retrofit are being done actively. However, the significant buildings built in the post-war reconstruction period, including many private buildings, as the existing ineligible structures. Under such circumstances, an important challenge of concrete buildings, with extremely low compressive strength of concrete, is how do evaluate and how to make seismic evaluation and retrofit. Although, concrete has been widely used as the main structure material in architectural and civil engineering structures, the performance of concrete be sensitive to lack of design and construction. And then, constructed in the immature time of mix proportion design of concrete and mixed technology focused on the problem of low-strength of concrete. In recent year, study on seismic performance of reinforced concrete structure with low-strength concrete has been done.

On the other hand, require a seismic evaluation for large-scale facilities, such as hospital, hotel, where are used by unspecified people. For this reason, “Amendment of the Act on Promotion of the Earthquake-proof Retrofit of Buildings” was enacted in Japan (2013). Because, it is needed to improve the earthquake resistant of various large-scale facilities, will be served as emergency condemnation of vulnerable, promulgated as the laws and regulations. In the past, by the administrative guidance, concrete buildings of 7-8 stories or more were adopted in steel reinforced concrete structure. However, filling of concrete in steel reinforced concrete structure has been difficult by the present of steel. Thus, there is a possibility that the existing buildings with the low-strength concrete in steel reinforced concrete structures are more in reinforced concrete structure.

In this research, for focusing on the existing large-scale facilities of steel reinforced concrete structure, we carried out the loading test of steel reinforced concrete columns of low-strength concrete, below the coverage of the existing seismic performance evaluation standard [1]. Main discussion is concentrated on the structural performance such as ultimate strength and deformation capacity.

2. Experimental work

2.1 Test program

Test program is shown in Table 1. The test specimens are shown in Fig.1. The test specimens are planned as assumption steel reinforcement concrete columns that were provided shear failure was preceded to be flexural failure. There were a total of 10 specimens of steel reinforcement concrete columns, five were full-web type steel and others were open-web type steel. The full-web type steels were welded the H-shaped steel. And, the open-web type steels were assembled in angle steel and batten plates.

Table 1 – Test Program

Specimens	F_c	Steel Type	$M/(Qd)$	N	N_u	${}_sN_u$	${}_mN_u$	${}_cN_u$	Steel (${}_sp_t$)	Batten plate (${}_sp_w$)	Main Reinforcements (${}_rp_t$)	Shear Reinforcements (${}_rp_w$)	Test series
18FC1515	18	Full-web Type	1.5	1095	3896	1675	502	1719	2H-200×100×5.5×8 (0.89%)	—	12-D13 (0.84%)	D6@100	I
09FC1530	9			1129	3728	1795	556	927				φ6@100 (0.19%)	II
09FC1515				994									
18FC1015	18		1.0	1094	3914	1675	502	1737				D6@100 (0.21%)	III
09FC1030	9			1172	3293			1116					
18BC1515	18	Open-web Type (Batten plate)	1.5	947	3537	1400	490	1647	8L-50×50×6 (1.25%)	PL6-30@200 (0.30%)	12-φ13 (0.87%)		I
09BC1530	9			956	2888	1454	480	954				φ6@100 (0.19%)	II
09BC1515				821									
18BC1015	18		1.0	982	3645	1454	490	1701				D6@100 (0.21%)	III
09BC1030	9			1046	3033			1089					

F_c:Design standard strength of concrete(N/mm^2), $M/(Qd)$:Shear span ratio

N:Axial force (kN), $N_u = {}_sN_u + {}_mN_u + {}_cN_u$

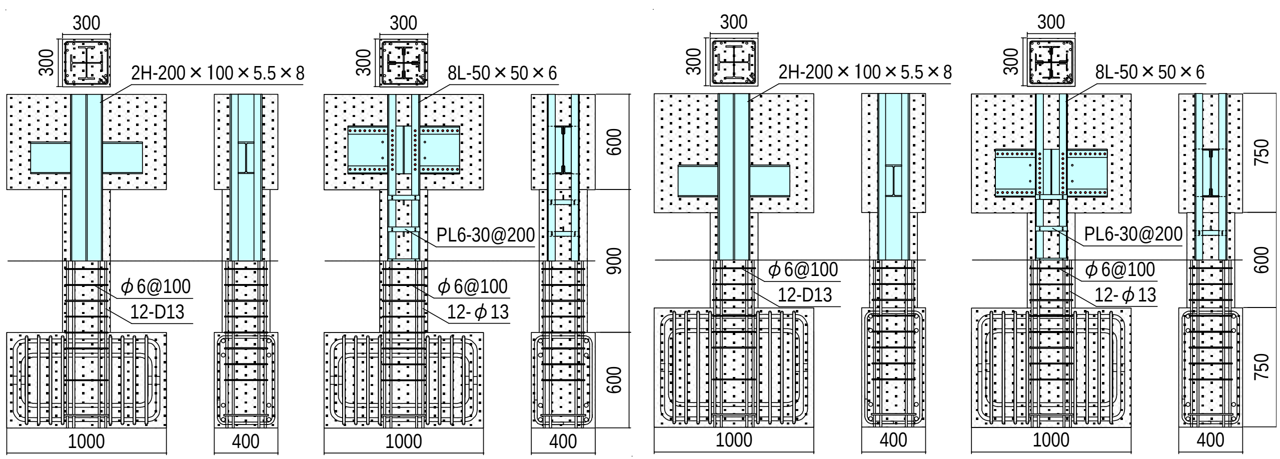
${}_sN_u$:Compression strength of steel(kN),${}_mN_u$:Compression strength of main reinforcements(kN),${}_cN_u$:Compression strength of concrete(kN)

${}_sp_t$:Tensile steel ratio, ${}_sp_w$:Batten plate ratio, ${}_rp_t$:Tensile reinforcement ratio, ${}_rp_w$:Shear reinforcement ratio

Column cross-section of the test specimens having a full-web type steel was 300mm×300mm. Steel was using the H-200×100×5.5×8, the band plate is not provided. Main reinforcements were used 12-D13, shear reinforcements in series I and III were D6@100, and in series II was ϕ6@100. After recommending of the AIJ standard for steel reinforced concrete structures, 3rd Edition [2], that had been revised in 1975, the construction had been building to use the full-web type steel and deformed steel bar (main reinforcements) in steel reinforcement concrete structures were be seemed to be the majority.

Also for the test specimens having an open-web type steel, column cross-section was 300mm×300mm. Steel was assembled into a batten shape by using the 8L-50×50×6 angle steel and batten plate of PL6-30@200, the band plate was not provided. Main reinforcements were applied round steel 12-ϕ13, shear reinforcements in series I and III were D6@100, series II were ϕ6@100. Steel reinforcement concrete structures had been using open-web type steel and round steel (main reinforcements) were seemed to be in many buildings, which were built before 1975. It should be noted that, in the buildings of the assumed age, the joint of angle steel and batten plate was used by rivet, but in this experiment , joined by using a hexagon socket head bolts of 2-M6 (pore size ϕ7).

The shear span ratio M / (Qd) was 1.5 in series I and series II, and 1.0 in series III.



Full-web type　　Open-web type　　Full-web type　　Open-web type

(a) $M/(Qd)$=1.5　　(b) $M/(Qd)$=1.0

Fig. 1 –Test specimens

For the specimens of concrete design strength F_c of 18N/mm^2 were placing both the column part and the stub part in together the concrete of 18N/mm^2. And, it was used the different strength of concrete in the test of the columns with the concrete design strength of 9N/mm^2. As the column portion was placed the concrete of 9N/mm^2, and the upper and lower stub of those were set to the concrete design strength of 24N/mm^2.

The mixing of concrete is shown in Table 2, and then, the material test results list of concrete and steel used in the specimens are shown in Table 3 and Table 4, respectively.

Table 2 – Mixing of concrete

F_c (N/mm^2)	Water (kg/m^3)	Cement (kg/m^3)	Fine aggregate (kg/m^3)	Coarse aggregate (kg/m^3)	Admixture (kg/m^3)	Water-cement ratio (%)	Fine aggregate ratio (%)
9	197	179	1058	806	1.29	110	57.3
18	194	244	1001	818	1.76	79.5	55.8
24	190	325	873	892	2.34	58.5	50.3

Table 3 – Material strength of concrete

Specimen	F_c (N/mm^2)	σ_B (N/mm^2)	σ_t (N/mm^2)	E_c (N/mm^2)
18FC1515	18	19.1	2.01	27707
09FC1530	9	10.3	1.35	20407
09FC1515	24	24.2	2.22	28355
18FC1015	18	19.3	1.86	25159
09FC1030	9	12.4	1.36	19549
	24	29.8	2.70	30456
18BC1515	18	18.3	1.98	27534
09BC1530	9	10.6	1.36	19157
09BC1515	24	25.4	2.32	28637
18BC1015	18	18.9	2.07	26601
09BC1030	9	12.1	1.35	21809
	24	29.4	2.29	29749

σ_B : compression strength, σ_t : tensile strength, E_c :Young modulus

Table 4 – Material strength of steel

Material	Test series I			Test series II			Stest sries III		
	σ_y (N/mm^2)	σ_u (N/mm^2)	elongation (%)	σ_y (N/mm^2)	σ_u (N/mm^2)	elongation (%)	σ_y (N/mm^2)	σ_u (N/mm^2)	elongation (%)
Flange	321	448	19.4	326	450	25.0	321	448	19.4
Web	307	435	21.4	347	464	23.2	307	435	21.4
Angle steel	310	446	19.1	322	467	25.7	322	467	25.7
Batten plate	269	360	30.3	269	360	30.3	269	360	30.3
Main reinforcement D13	330	470	18.4	366	520	21.1	330	470	18.4
Main reinforcement φ13	307	430	26.0	301	417	34.2	307	430	26.0
Shear reinforcement φ6,D6	346	524	20.9	379	542	18.6	346	524	20.9

σ_y: yield strength, σ_u :tensile strength

Compression axial force N for the specimen of F_c=18N/mm^2 was calculated by $N=0.15{}_cN_u+0.5{}_sN_u$ (where, ${}_cN_u$ is the compression strength of the concrete, ${}_sN_u$ is the compressive strength of the steel) on columns that had been designed in 18FC1515, 18FC1015, 18BC1515, and 18BC1015. And so, $N=0.3{}_cN_u+0.5{}_sN_u$ was based on F_c=9N/mm^2 specimens such as 09FC1530, 09FC1030, 09BC1530 and 09BC1030. Moreover, for the purpose of comparison that were subjected of $N=0.15{}_cN_u+0.5{}_sN_u$ that was planning for 09FC1515 and 09BC1515 in F_c = 9N/mm^2 specimens.

2.2 Loading method

All the specimens were subjected on the same loading cycle. The loading system is shown in Fig. 2.The height of the inflection point of the columns were 450mm and 300mm, while shear span ratio $M/(Qd)$ of columns were 1.5 and 1.0, respectively. After fixing the specimens in the loading bed, loaded a constant compression axial force N and repeated-reverse symmetric moment to column portion. At first, under the displacement control, an amplitude of rotation angle R=±0.25%rad was carried out a cycle. After that, rotation angle was changed in gradual increasing of amplitude by R=±0.5%rad and carried out two cycle on every angle. The experiment was terminated with up to rotation angle R=±5.0%rad of amplitude.

Measurement of displacement was measured the relative horizontal displacement δ_u between upper stub and lower stub. And then, measurement of stain was measured by stain gauges on main reinforcements, shear reinforcements, H-shaped steel's flange and web, and angle steel and batten plate, respectively.

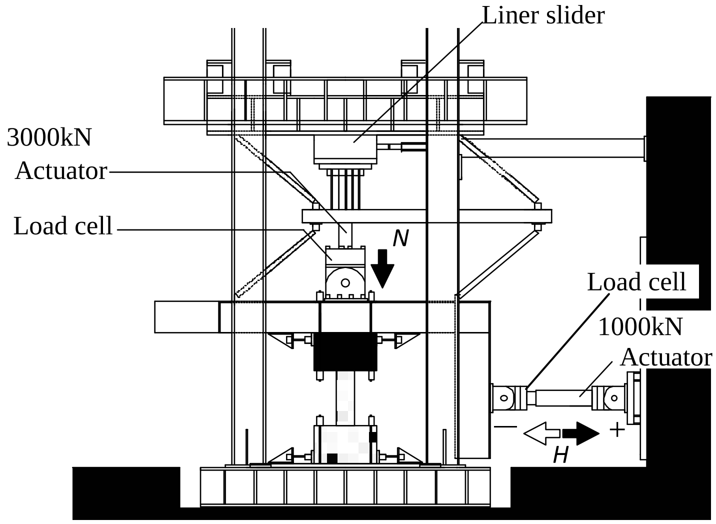


Fig. 2 –Loading System

3. Experimental results

The hysteresis loops of specimens are shown in Fig. 3. The dotted line in the Fig. 3 means the calculated value of ultimate shear strength recognized in the seismic performance evaluation standard [1]. Fig. 4 show the failure state of low-strength concrete specimens 09FC1530 and 09BC1530, when amplitude was rotation angle R=±1.0%rad.

3.1 Full-web type

In this type, when rotation angle R=±0.25%rad carried out, diagonal cracks occurred at the center of column. Then, gradual increase of rotation angle made diagonal cracks become expending to the both terminal of test column. When amplitude of rotation angle R=±1.0%rad, adhesion cracks started occurring along the main rebar and strong axis of flange. The maximum strength arrived at rotation angle R=±1.0 and ±1.5%rad. After maximum strength, diagonal cracks and adhesive cracks increased and expanded widely on the whole surface. However, the hysteresis loop of this steel type maintained the spindle shape until the end of the experiment. Moreover, shear reinforcement yield was occurred when reaching the maximum strength, except 09FC1030 and 09FC1515 specimens.

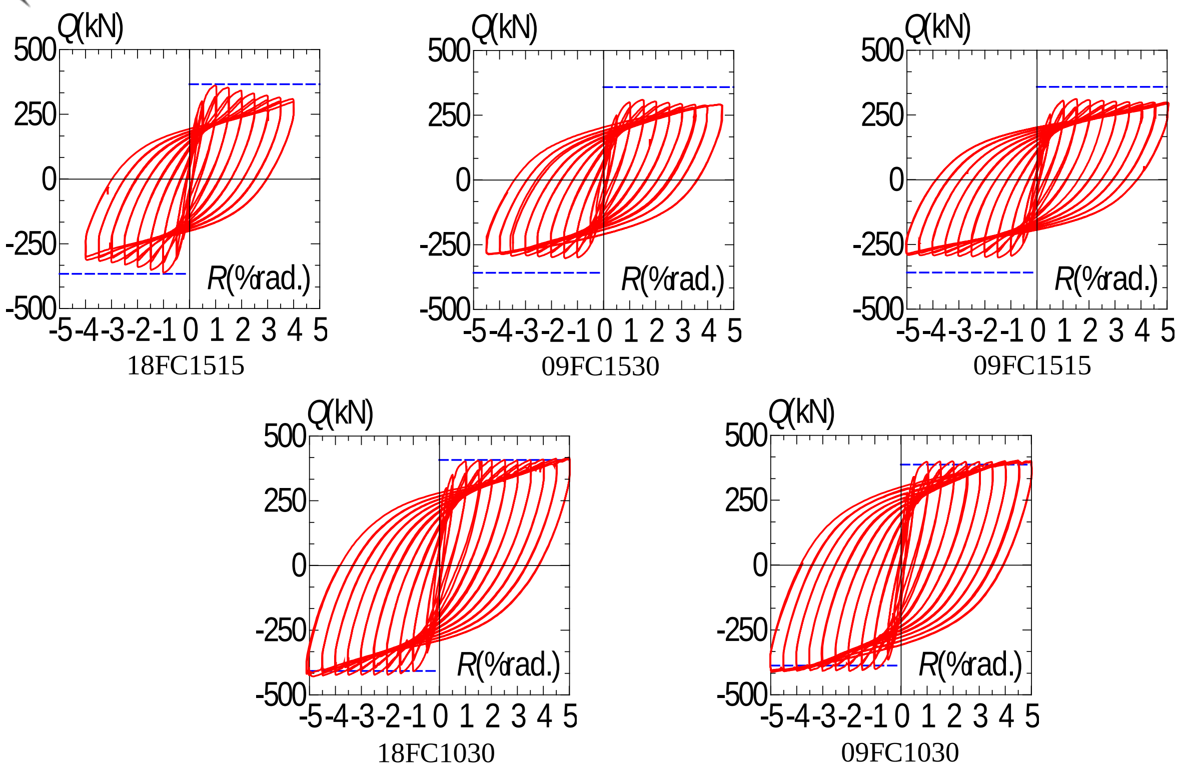
3.2 Open-web type

The failure state of open-web type column was similar to the full-web type test column. As the above, first, it also occurred diagonal cracks at the center, and then, expanded to the both end sides. The maximum strength reached at rotation angle R=±1.0%rad for all open-web specimens. After maximum strength, the hysteresis loop became slip characteristics prominently. Moreover, shear reinforcement yield was also occurred in open-web type specimens when reaching the maximum strength, except 09BC1530 and 09BC1515 specimens.

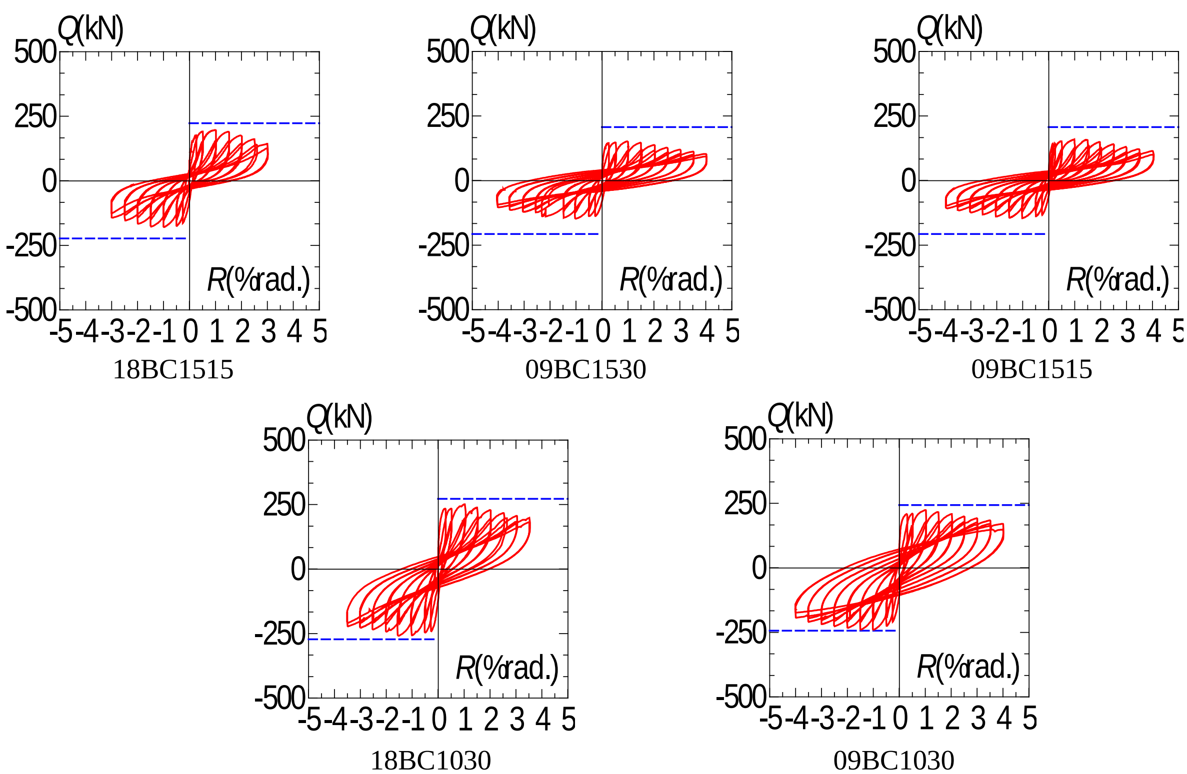
3.3 Comparison of experimental results

Comparison between experimental results of full-web type steel column and open-web type steel column, the aspects of failure state were seemed almost the same.

On the other hand, the maximum strength, strength decreased situation after maximum strength and the shape of hysteresis loop were obviously difference. Despite using the low-strength concrete, full-web type steel reinforced concrete column had been confirmed to have the excellent deformation capacity. There was also confirmed that the cause of maximum strength was depended mainly on the strength of concrete and slightly depended on the axial force.



(a) Full-web type



(b) Open-web type

Fig. 3 –The hysteresis loop

09FC1530

(a) Full-web type

09BC1530

(b) Open-web type

Fig. 4–Failure state of R=1.0%rad.

4. Ultimate shear strength

The relationship between experimental values and calculated values are presented in Table 5 and Fig. 5.

4.1 JBDPA standard for existing steel reinforced concrete buildings

The ultimate shear strength is calculated with the equations applied on the JBDPA standard for structural calculation of existing steel reinforced concrete buildings [1]. In this standard, the evaluation of ultimate shear strength Q_{se} of steel reinforced concrete column is based on applicable coverage of the compressive strength of concrete is 13N/mm^2 and more. Eq. (1) and Eq. (2) are proposed the full-web type steel reinforced concrete column and the open-web type steel reinforced concrete column, respectively.

$$Q_{se}=\left\{\frac{0.053\,{}_rp_t^{\,0.23}\cdot k_{cs}\cdot(18+\sigma_B)}{M/(Q\cdot d)+0.12}+0.85\sqrt{{}_rp_w\cdot{}_r\sigma_{wy}}+0.1\sigma_0\right\}\cdot b\cdot{}_rj+{}_sQ_u \tag{1}$$

$$Q_{se}=\left\{\frac{0.053\,p_t^{\,0.23}\cdot(18+\sigma_B)}{M/(Q\cdot d)+0.12}+0.85\sqrt{{}_rp_w\cdot{}_r\sigma_{wy}+\frac{{}_sp_w\cdot{}_s\sigma_{wy}}{2}}+0.1\sigma_0\right\}\cdot b\cdot j \tag{2}$$

where, k_{cs} is reduction coefficient of direct shear fracture on flange, ${}_r\sigma_{wy}$ is yield strength of reinforcement, ${}_s\sigma_{wy}$ is yield strength of steel, b is width of column, ${}_rj$ is distance between tensile and compressive resultants of reinforced concrete, j is distance between tensile and compressive resultants, p_t is tensile steel materials ratio and σ_0 is axial stress of column. And, ${}_sQ_u$ in Eq. (1) is the minimum value of ultimate flexural strength ${}_sQ_{mu}$ and ultimate shear strength ${}_sQ_{su}$, when the axial force of steel portion set to "0". In that case, for the present experiment, the test specimens of shear span ratio of 1.5 resulted as ${}_sQ_{su}>{}_sQ_{mu}$, while shear span ratio 1.0 specimens had become ${}_sQ_{su}<{}_sQ_{mu}$.

In the case of depending on the Eq. (1) and Eq. (2), the experimental value Q_{exp} divided by calculated value Q_{se} were 0.73~1.06 while the average value was 0.92. Using Eq. (1) and Eq. (2), 8 specimens' experimental results were lower than calculated results. Including test specimens with normal strength of concrete F_c=18N/mm^2, 8 of 10 specimens were confirmed as instability condition.

Similar to previous studies of reinforced concrete member with low-strength concrete, the evaluation of steel reinforced concrete column resulted also lower than the calculation result. Thus, we applied another evaluation formulae from previous studies to obtain an analytical expression for the stability condition between calculated and experimental results.

4.2 Yamamoto’s Model

Frist comparison method was based on the previous study of reinforced concrete member with low-strength concrete. By Yamamoto’s model, it was evaluated the ultimate shear strength using a reduction coefficient k_r for confirming become a relevant equivalent [3]. In this paper also used this reduction coefficient k_r extended to the evaluation of steel reinforced concrete member in order to confirm the better results. Ultimate shear strength for full-web type column of steel reinforced concrete is given by Eq. (3), then ultimate shear strength for open-web type column of steel reinforced concrete is given by Eq. (4).

$$Q_{ym} = k_r \cdot \left\{ \frac{0.053\,{}_r p_t^{0.23} \cdot k_{cs}(18+\sigma_B)}{M/(Q \cdot d)+0.12} + 0.85\sqrt{{}_r p_w \cdot {}_r \sigma_{wy}} + 0.1\sigma_0 \right\} \cdot b \cdot {}_r j + {}_s Q_u \tag{3}$$

$$Q_{ym} = k_r \cdot \left\{ \frac{0.053\,p_t^{0.23}(18+\sigma_B)}{M/(Q \cdot d)+0.12} + 0.85\sqrt{{}_r p_w \cdot {}_r \sigma_{wy} + \frac{{}_s p_w \cdot {}_s \sigma_{wy}}{2}} + 0.1\sigma_0 \right\} \cdot b \cdot j \tag{4}$$

$$\text{where, } k_r = 0.244 + 0.056\sigma_B \qquad (\sigma_B \leq 13.5) \tag{5}$$

When the experimental value was divided by the calculated value of Yamamoto’s model, the results were 0.88~1.13 (the average value is 0.98). In spite of using Eq. (3) and Eq. (4) had been improved evaluation accuracy than the case of structural standard, it was confirmed that 7 of 10 specimens still in instability condition because experimental results were lower than calculated results.

4.3 Yasojima’s model

Next method also based on the previous study of reinforced member with low-strength concrete. By Yasojima’s model [4], the evaluation formula had been proposed that the strength of shear reinforcement was reduced by the strength of concrete using in the experiment time. As the above model, extended the Yasojima’s model to the JBDPA standard. Case of full-web type test column of steel reinforced concrete is calculated by Eq. (6) and the rest is Eq. (7).

$$Q_{ys} = \left\{ \frac{0.053\,{}_r p_t^{0.23} \cdot k_{cs}(18+\sigma_B)}{M/(Q \cdot d)+0.12} + \alpha_L\sqrt{{}_r p_w \cdot {}_r \sigma_{wy}} + 0.1\sigma_0 \right\} \cdot b \cdot {}_r j + {}_s Q_u \tag{6}$$

$$Q_{ys} = \left\{ \frac{0.053\,p_t^{0.23}(18+\sigma_B)}{M/(Q \cdot d)+0.12} + \alpha_L\sqrt{{}_r p_w \cdot {}_r \sigma_{wy} + \frac{{}_s p_w \cdot {}_s \sigma_{wy}}{2}} + 0.1\sigma_0 \right\} \cdot b \cdot j \tag{7}$$

$$\text{where, } \alpha_L = 0.038\sigma_B \leq 0.85 \qquad (\sigma_B \leq 22) \tag{8}$$

When the experimental value was divided by the calculated value of Yasojima’s model, the results became 0.88~1.14 (the average value is 0.99). Those were almost the same results of Eq. (3) and Eq. (4), however, also 6 specimens still in the instability condition as same as above reason.

4.4 AIJ Standard for steel reinforced concrete structures

In 6th edition of the AIJ standard for steel reinforced concrete structures [5], based on the applicable coverage for the compression strength of concrete is 18N/mm^2 and more, the formula of ultimate shear strength Q_{aij} is given by Eq. (9).

$$Q_{aij} = \min({}_r Q_{su}, {}_r Q_{mu}) + \min({}_s Q_{su}, {}_s Q_{mu}) \tag{9}$$

In Eq (9), ${}_r Q_{su}$ and ${}_r Q_{mu}$ are the strength determined by shear and flexure which considered on reinforced section. On the other hand, ${}_s Q_{su}$ and ${}_r Q_{su}$ are the strength determined by shear and bending which considered on steel section. Moreover, by the explanation of this standard, using truss-arch model which is based on plasticity theory for the evaluation of ${}_rQ_{su}$, because the strength of concrete is not affected by the scope of application. The truss-arch model is given by Eq. (10).

$$ {}_rQ_{su} = b \cdot j_t \cdot {}_rp_w \cdot {}_r\sigma_{wy} \cdot \cot\phi + \tan\theta\,(b - \beta' \cdot b')\cdot \mu \cdot D \cdot \frac{\sigma_B}{2} \tag{10} $$

$$ \tan\theta = \sqrt{\left(\frac{l'}{D}\right)^2 + 1} - \frac{l'}{D} \tag{11} $$

$$ \beta' = (1 + \cot^2\phi) \cdot \frac{b'}{b} \cdot \frac{{}_rp_w \cdot {}_r\sigma_{wy}}{\sigma_B} \tag{12} $$

$$ \mu = 0.5 + \frac{b'}{b} \leq 1.0 \tag{13} $$

where ϕ (45°) is angle of concrete compression of truss mechanism, θ is angle of concrete compression of arch mechanism, and *b'* is effective width of concrete and the rest notations are intended to refer to the AIJ standard for steel reinforced concrete standard , 6th edition[5].

Furthermore, it is not subjected open-web type steel reinforced concrete column to evaluate in 6th edition, the evaluation of ${}_sQ_{su}$ is calculated by the formula in the 5th edition of the AIJ standard for steel reinforced concrete standard [6]. In this study, the result was out ${}_rQ_{su} < {}_rQ_{mu}$ for all test specimens. Then, it was resulted that ${}_sQ_{su} > {}_sQ_{mu}$ for specimens with shear span ratio of 1.5 and ${}_sQ_{su} < {}_sQ_{mu}$ for shear span ratio of 1.0 of full-web type, but all of open-web type had become ${}_sQ_{su} < {}_sQ_{mu}$.

The ratio of experimental values Q_{exp} by calculated values Q_{aij} were 1.02~1.29, with average amount of 1.12. In this method, all experimental values exceeded the calculated values. So, it was confirmed that can evaluate the lower limit of the ultimate shear strength for steel reinforced concrete column.

4.5 Consideration for the calculated values of ultimate shear strength

In the case of using the AIJ standard for steel reinforced concrete structure on low-strength concrete model, most of the experimental values were lower the calculated values. It can be considered as the concrete became failure before shear reinforcement yeild. And it was also reason, when the axial force of steel portion in low-strength concrete became greater, the ultimate flexure strength of steel became lower.

In addition, despite using the reinforced concrete based formula, such as Yamamoto's model and Yasojima's model, there were many cases left in instability condition in steel reinforced concrete members. It can be considered that it would be had influenced by specific adhension cracks occuring along the steel flange side.

Table 5 – Experimental and calculated results

Test body	Q_{exp}(kN)	Q_{se}(kN)	Q_{ym}(kN)	Q_{ys}(kN)	Q_{aij}(kN)	Q_{exp}/Q_{se}	Q_{exp}/Q_{ym}	Q_{exp}/Q_{ys}	Q_{exp}/Q_{aij}
18FC1515	362	366	366	358	327	0.99	0.99	1.01	1.11
09FC1530	310	359	328	330	286	0.86	0.95	0.94	1.08
09FC1515	322					0.90	0.98	0.98	1.12
18FC1015	429	408	408	400	394	1.05	1.05	1.07	1.09
09FC1030	412	387	363	362	333	1.06	1.13	1.14	1.24
18BC1515	197	223	223	212	191	0.88	0.88	0.93	1.03
09BC1530	152	207	173	173	138	0.73	0.88	0.88	1.10
09BC1515	159					0.77	0.92	0.92	1.15
18BC1015	258	272	272	263	254	0.95	0.95	0.98	1.02
09BC1030	240	244	225	217	186	0.98	1.07	1.11	1.29

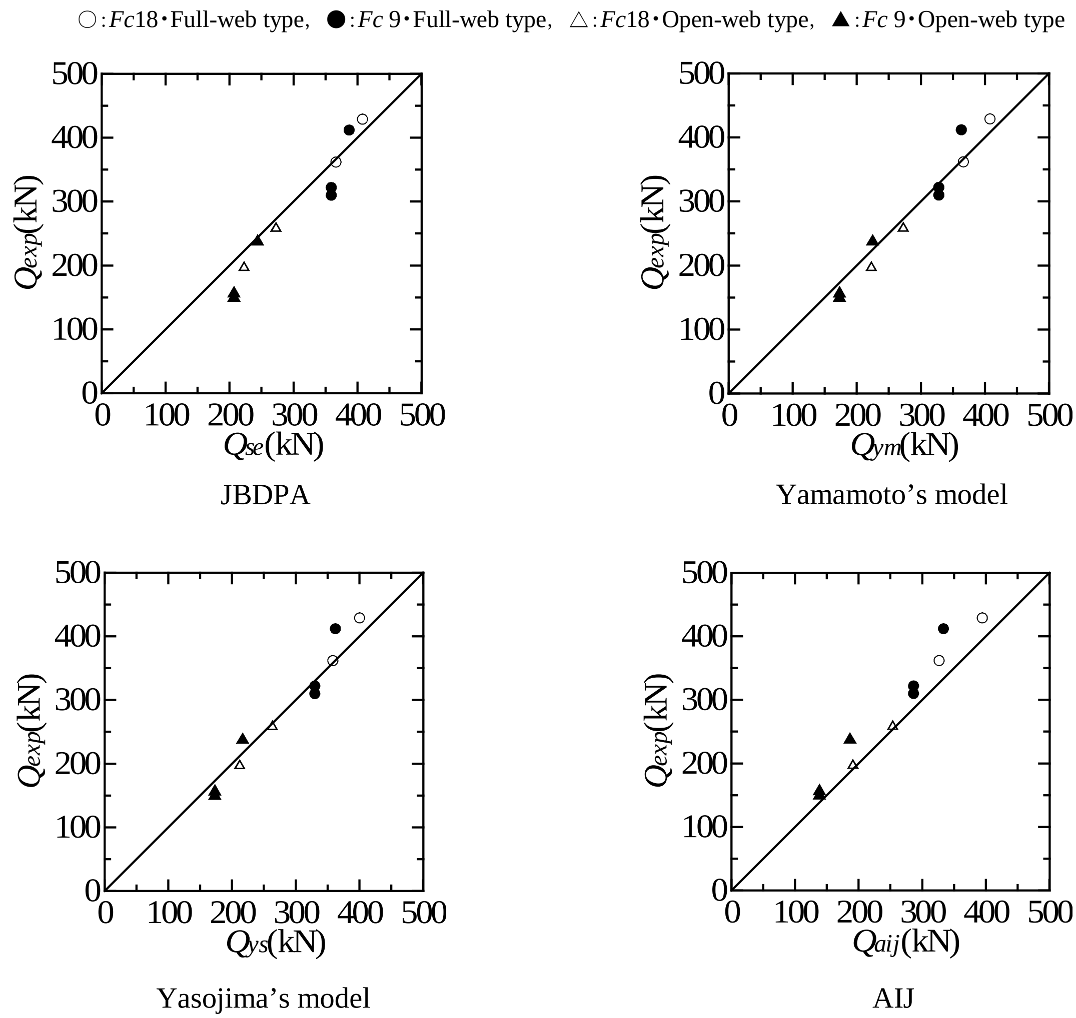


Fig. 5 – Experimental and calculated values

5. Conclusions

The results obtained on the present study are concluded as below.

1) Although occurred the same failure state, the maximum strength, the behavior after the attainment of the maximum strength, and the hysteretic differed widely, depending on the type of steel to be built. Even in the case of low-strength concrete, it can be confirmed that the full-web type steel columns possessed the excellent deformation capacity.

2) JDBPA seismic diagnostic standard to ultimate shear strength cannot be evaluated for the steel reinforced concrete columns with low-strength concrete. Moreover, in spite of using the reduction factor that had been used for reinforced concrete members, there was also evaluated that the results in dangerous situation for the steel reinforced concrete columns with low-strength concrete.

3) On the other hand, the ultimate shear strength can be estimated the equation based on the theory of truss and arch mechanism in AIJ design for the steel reinforced concrete standard with low-strength concrete.

6. Acknowledgements

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