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SEISMIC BEHAVIOR OF GIRDER-TO-COLUMN CONNECTIONS DEVELOPED FOR AN ADVANCED MIXED STRUCTURE SYSTEM

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SUMMARY

A mixed structural system, introduced in this paper, comprised of reinforced concrete columns and structural steel girders which utilizes both the advantages of steel and reinforced concrete. Experiments on the two types of joint-panel details, the full-flange-type panel and the tapered-flange-type panel, were carried out by using one half scaled models of cruciform girder-to-column subassemblages with short transverse girders at the joint-panels. Five sets of test specimens were provided. Results of the experiments show the high feasibility to the practical use of the proposed mixed system including the details of girder-to-column connections developed here.

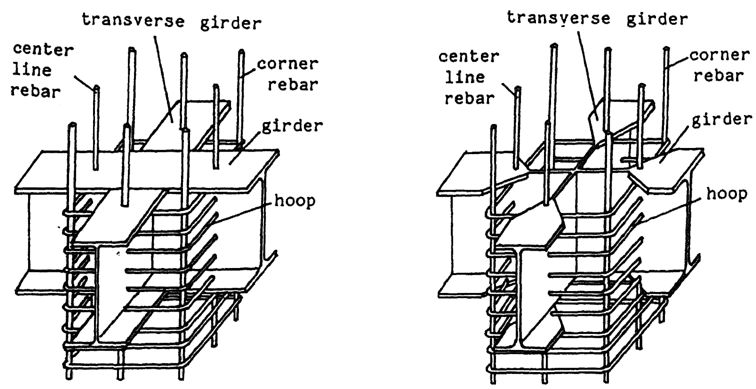
INTRODUCTION

Compared with reinforced concrete structures, steel structures have their own advantages in weight, ductility, span length, term of the construction and so on. However, they are not always more competitive in the total construction cost. On the other hand, reinforced concrete structures have their advantages in high stiffness and low cost. By combining the both advantages of steel and reinforced concrete, mixed structural system will be widely used in the future. Introduced in this paper is the proposed mixed structural system which comprised of reinforced concrete column and structural steel girders, including the details of girder-to-column connections developed here. This mixed system is selected as one example among possible mixed systems (Ref. 2).

In order to apply this mixed system to actual structures, discussed in this paper are the ultimate strength and the hysteretic behavior of girder-to-column connections for lateral forces.

JOINT-PANEL DETAILS

Typical joint-panel details are shown in Fig. 1. Depicted are the details of the full-flange-type panel, in which two perpendicular structural steel H-shaped girders penetrate the reinforced concrete column, see Fig. 1(a), with the main reinforcing bars (rebars) in the column corners passing through the panel zone, while center line rebars are welded to the top and bottom of the steel girders. The hoops in the panel zone are welded to the top or bottom flanges, or the web plates of the steel girders. Fig. 1(b) shows the details of the tapered-flange-type panel, in which girder flanges are tapered by cutting. The taper angle measures 45 degrees. These cut girder flanges assure reliable concrete casting into the panel zone.



(a) Full-flange-type (b) Tapered-flange-type

Fig. 1 Details of Joint-panels

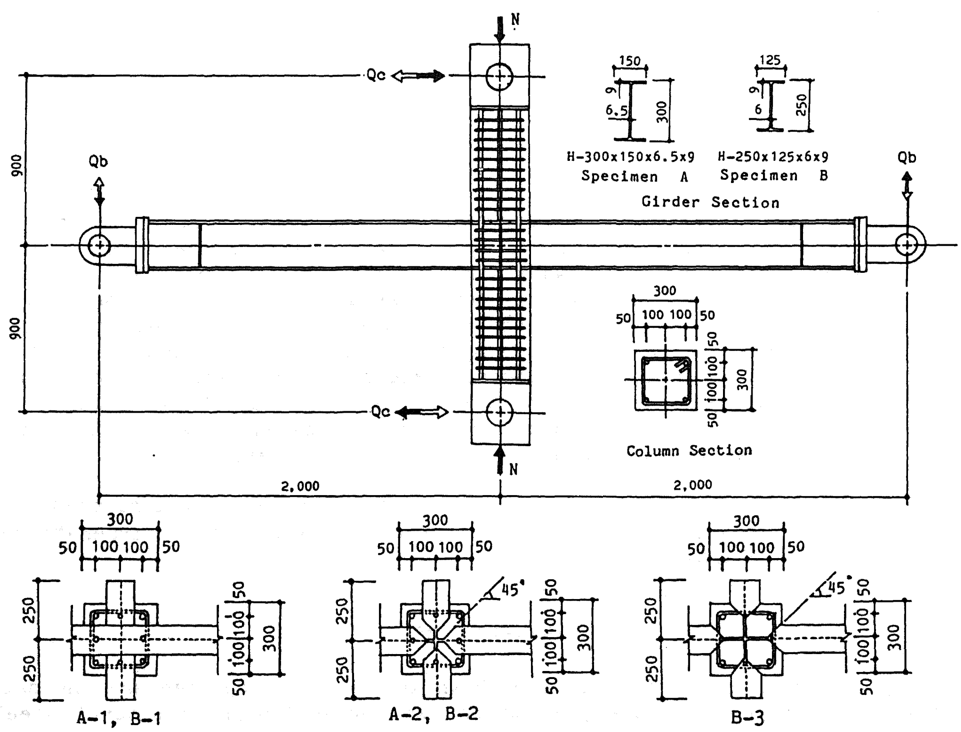


Fig. 2 Shape of Specimens

Table 1 Mechanical Properties

Specimen		Steel		Reinforcing Bar			Concrete
		σy	σt	Size	σy	σt	σc
A-1	Flange	31.0	45.8	D16	35.6	51.8	2.12
	Web	33.6	47.1	D13	37.7	55.4	
A-2	Flange	30.1	43.6	D16	35.0	52.2	2.17
	Web	33.8	45.2	D13	35.9	50.6	
B-1	Flange	34.7	50.3	D16	35.6	51.8	2.10
	Web	37.5	51.5	D13	37.7	55.4	
B-2	Flange	33.1	44.8	D16	35.0	52.2	2.19
	Web	38.6	47.2	D13	35.9	50.6	
B-3	Flange	33.1	44.8	D16	35.0	52.2	2.15
	Web	38.6	47.2	D13	35.9	50.6	

(unit : MPa)

TEST SCHEME

The experiments on the two types of joint-panel details, the full-flange-type panel and the tapered-flange-type panel, were carried out by using one half scaled models of cruciform girder-to-column subassemblages with short transverse girders at the joint-panels. Five sets of test specimens, two sets of specimens for the full-flange-type panels and three sets of specimens for the tapered-flange-type panels, were provided. The shape of the specimens is shown in Fig. 2. The ratio of the strength of columns to that of girders and the amount of the flange cutting of the steel girders in the panel zone were selected as test parameters. The mechanical properties of steel, rebar and concrete are shown in Table 1. The specimens, whose columns are weaker than girders, are denoted by "A" and the specimens with strong column and weak girders are affixed with "B". The specimens with full-flange-type panels are denoted by "1", those with tapered-flange-type panels whose taper started from the rebar location are denoted by "2" and the specimen with the taper starting at the column face is denoted by "3" (see Fig. 2).

Both the ends of the girder were loaded inversely by two actuators simulating seismic forces with a constant axial column load of 620kN, as shown in Fig. 2. Well-defined cyclic and reversed loading was applied to the specimens, which induced severe shear stresses in the panel zone.

TEST RESULTS

First, flexural cracks were observed in the columns followed by subsequent diagonal cracking in the panel zone. Then, the shear yielding of the web plate of the steel girder in the panel zone occurred. Finally, in case of the specimens with the full-flange-type panels ("1"), the center line rebars fractured close to the weld point on the top of the girder flange because of poor workmanship of the weld execution. This fracture brought about the spalling of the cover concrete in the panel zone. On the other hand, in case of the specimens with the tapered-flange-type panels ("2" & "3"), the center line rebars did not fracture, forces were transmitted properly from steel girder to reinforced concrete column and the yielding of the tensile reinforcement occurred. Only minor spalling of the concrete cover was observed. In all specimens, the hoops in the panel zone confined well the core concrete up to the ultimate stage of loading, and the stress of the hoops was less than one half of the yield stress.

The hysteresis curves of the column shear force (Qc) vs. story drift angle (R) relationships are shown in Fig. 3. Each specimen showed quite stable loops. However, as the amount of cutting of the steel flange increases in the panel zone (tapered-flange-type panels), the maximum strength decreases. The reduction of the maximum strength is associated with the reduction of local bearing force on the steel flange and thus the specimens with tapered-flange-type panels are associated with a reduction of the spalling cracks of the column concrete cover. Severe deterioration of load carrying capacity was observed in specimens with the full-flange-type panels (A-1 & B-1) at the drift angle of 0.05 radians, where the severe spalling of cover concrete was observed in the panel zone because of the fracture of center line rebars. The ductility was quite large in all specimens and it did not deteriorate at least up to a story drift angle of 0.05 radians, while critical shear deformation angle at the panel zone corresponding to this story drift state was 0.025 radians. Without the fracture of center line rebars, the strength deterioration would never have occurred in specimens with the full-flange-type panels at the critical story drift angle. Thus, the welding of center line rebars to the top of the girder flange must be well executed to eliminate the severe strength deterioration caused by the fracture of rebar.

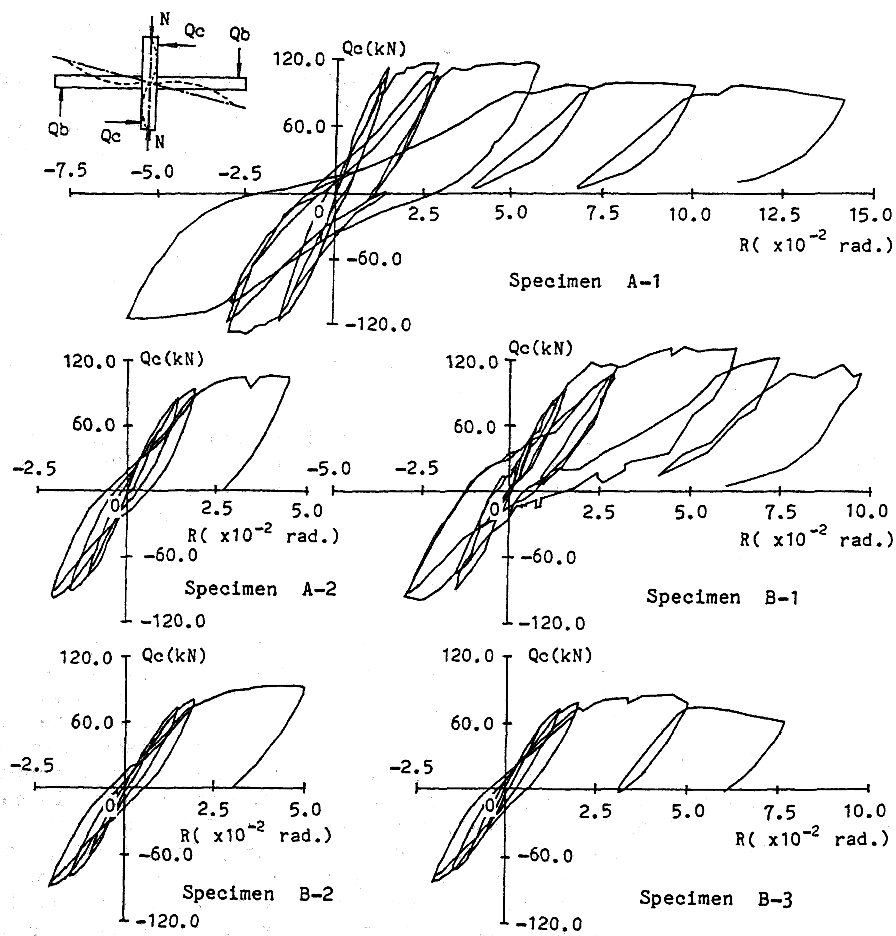


Fig. 3 Hysteresis Curves (Qc - R Relationships)

Table 2 Comparison of Diagonal Crack Strengths in Panel Zone

Specimen	eQcr	Qcr	eQcr/Qcr
A-1	87.3	32.6	2.68
A-2	66.6	33.4	1.99
B-1	75.3	25.5	2.95
B-2	65.8	26.5	2.49
B-3	51.4	26.1	1.97

(unit : kN)

* Strengths in the above table are shown as column shear force.
eQcr : Measured diagonal crack load in panel zone.
Qcr : Diagonal crack strength in panel zone estimated as $\tau({}_cb \cdot {}_{mc}d + 15\,{}_jt_w \cdot {}_{sc}d)$, τ=0.1Fc and ${}_{sc}d = {}_{mc}d$.

DISCUSSION

The calculated results for the crack (diagonal cracking) initiation strengths of joint-panels are compared with the test ones for each specimen in Table 2. The test results are two or three times higher than the calculated ones. The shear stress (τ) at the onset of diagonal cracking is taken as 0.1Fc in Ref. 1 (see Table 2). Therefore, the experiments show that the crack initiation shear stress can be considered to be 0.3Fc in the case of full-flange-type panels and 0.2Fc in the case of tapered-flange-type panels.

As for the ultimate shear strength of the joint-panels, the procedure described in Ref. 1 is assumed to be applicable to this case,

$$jMu1 = cVe\ (\ jFs \cdot j\delta + wP \cdot w\sigma y\ ) + 1.2\ sV \cdot s\sigma y\ /\ \sqrt{3} \tag{1}$$

in which $cVe = ({}_cb/2) \cdot {}_{sb}d \cdot {}_{mc}d$,

$sV = {}_jt_w \cdot {}_{sb}d \cdot {}_{mc}d$,

Fc : nominal design strength of concrete (MPa),
jFs : concrete shear strength which is smaller value of 0.12Fc or 1.76 + (3.6Fc/100) (MPa),
$j\delta$: coefficient dependent on the joint shape (cross-shaped = 3),
wP : reinforcement ratio of hoops,
${}_cb$: width of column (mm),

$w\sigma y$: tensile yield strength of hoops (MPa),
${}_{mc}d({}_{sb}d)$: distance from centroid of compression steel to that of tension in column (girder) (mm),
$s\sigma y$: tensile yield strength of the structural steel (MPa),
${}_jt_w$: thickness of steel web panel (mm).

Taking the stress of hoops measured in the experiments into account, eq.(1) can be represented as follows.

$$jMu2 = cVe\ (\ jFs \cdot j\delta + (\ wP \cdot w\sigma y/2\ )) + 1.2\ sV \cdot s\sigma y/\sqrt{3} \tag{2}$$

Then, taking the width of effective panel concrete into account, eq.(2) can be further modified as follows.

$$jMu3 = cVe1 \cdot jFs \cdot j\delta + cVe2 \cdot (\ wP \cdot w\sigma y/2\ ) + 1.2\ sV \cdot s\sigma y/\sqrt{3} \tag{3}$$

Table 3 Comparison of Ultimate Shear Strengths

Specimen	eQm	jQu1	eQm/jQu1	jQu2	eQm/jQu2	jQu3	eQm/jQu3
A-1	125.1	116.8	1.07	101.2	1.24	101.2	1.24
A-2	105.7	115.8	0.91	101.0	1.05	101.0	1.05
B-1	130.2	97.9	1.33	85.0	1.53	82.1	1.59
B-2	92.4	98.2	0.94	85.9	1.08	82.9	1.11
B-3	85.4	98.0	0.87	85.8	1.0C	82.7	1.03

(unit : kN)

* Strengths in the above table are shown as column shear force.
eQm : Measured maximum load.
jQu1 : Ultimate shear strength in panel zone calculated from eq.(1).
jQu2 : Ultimate shear strength in panel zone calculated from eq.(2).
jQu3 : Ultimate shear strength in panel zone calculated from eq.(3).

in which $cVe1 = {}_bb \cdot {}_{sb}d \cdot {}_{mc}d$,

$cVe2 = ({}_cb - {}_bb) \cdot {}_{sb}d \cdot {}_{mc}d$,

${}_bb$: width of girder (mm)

The calculated results for the ultimate shear strengths are compared with the test ones for each specimen in Table 3. Making use of eq.(1), the ratios of the experimental maximum strength to the calculated one for only the tapered-flange-type joints are smaller than unity. However, making use of eq.(2) or eq.(3), the ratios of the experimental maximum strength to the calculated one are all larger than unity. This means that tapered-flange-type joints do not satisfy the ultimate shear strength criteria in the panel zones (eq.(1)) required by the SRC Standard of AIJ (Ref. 1). Thus, taking the actual stress of hoops and the width of effective panel concrete into account for the SRC Standard of AIJ , even tapered-flange-type joints well satisfy the ultimate shear strength criteria in the panel zones.

CONCLUDING REMARKS

The strength of the joint-panel decreases as the girder flange is cut in the panel zone because of the reduction of local bearing force on the steel flange. Herein, the strength satisfies the new criteria modifying the SRC Standard of AIJ (Ref. 1) by taking the measured stress of hoops and the width of effective panel concrete into account. On the other hand, the ductility was quite large and it did not deteriorate at least up to a story drift of 0.05 radians, while critical shear deformation angle at the panel zone corresponding to this story drift state was 0.025 radians.

The above conclusions show the high feasibility to the practical use of the proposed mixed structure system including the details of girder-to-column connections developed here.

REFERENCES

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