Elasto-plastic behavior of beam-to-column connections with fillets of steel bridge frame piers

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Abstract

The seismic design method of beam-to-column connections of steel bridge frame piers has not been established and it is necessary to develop it. Moreover the fatigue damage of beam-to-column connections of steel bridge frame piers is one of serious problems. It is proposed to install the fillets in beam-to-column connections of steel bridge frame piers for improving fatigue performance. The influence of fillets on seismic performance of beam-to-column connections must be considered in developing the seismic design method. In this study, cyclic loading experiments were conducted. Based on the experimental results, the elasto-plastic behavior of beam-to-column connections with fillets and the influence of fillets on elasto-plastic behavior of beam-to-column connections were investigated for developing the seismic design method in the future works.

1 INTRODUCTION

The Hyogo-ken Nanbu Earthquake occurred in January 1995, and it caused destructive damages to highway bridges like never seen before in Japan[1)]. The specifications for highway bridges were revised in 1996[2),3)] in consideration of the damage and the ductility design method was introduced to steel bridge piers. And the more detailed seismic design method for steel bridge piers was specified in the 2002 specifications[4),5)]. Besides the specifications for highway bridges, some methods for estimating the seismic performance of steel bridge piers or steel members have been already proposed[6)].

On the other hand, seismic design method of beam-to-column connections of steel bridge frame piers has not been established because the elasto-plastic behavior has not been investigated sufficiently and there are a lot of things to make clear for developing the ductility design method. Then beam-to-column connections of steel bridge frame piers are sometimes applied to elastic design method against level 2 Earthquake Ground Motions and it is not rational. It is necessary to clear up the elasto-plastic behavior of them in order to develop the ductility design method as the seismic design.

Moreover the fatigue damage of beam-to-column connections of steel bridge frame piers is one of serious problems. A lot of attempts have been conducted in order to improve the fatigue performance of beam-to-column connections. One of the methods for improving the fatigue performance is to install the fillets in beam-to-column connections. Fillets can reduce the high stress concentration by sheer lag at the corners in the beam-to-column connections and it can cause the improvement of the fatigue performance. However buckling at fillets is easy to occur and the buckling at fillets may have a bad influence on the seismic performance of beam-to-column connections. It is important to make clear the influence of fillets on the

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elasto-plastic behavior of beam-to-column connections.

In this study, cyclic loading experiments of beam-to-column connections of steel frame piers were carried out in order to grasp of the elasto-plastic behavior of beam-to-column connections with fillets and the influence of fillets on it.

2 OUTLINE OF EXPERIMENTS

2.1 Test Specimens

Two types of test specimens were employed. One of them is with fillets and another is without fillets. In this study, survey of major parameters of actual beam-to-column connections of steel frame piers which have been designed by the 1996 or the 2002 design specifications for high bridges were carried out so that the parameters of test specimens could be set in consideration of those of actual beam-to-column connections of steel bridge frame piers. Fig.1 shows the survey results of the parameters. In Fig.1, R_R and R_F are the with-thickness ratio parameters of plate panels between longitudinal stiffeners and overall stiffened plate panels respectively. γ_l^* is relative stiffness ratio of a longitudinal stiffener based on the elastic buckling theory. The definitions of parameters mentioned above are identical to those stipulated in the 2002 specifications[4),5)] and given as follows.

$$R_R = \frac{b}{t}\sqrt{12\frac{\sigma_y}{E}\frac{1-\nu^2}{4n^2\pi^2}} \tag{1}$$

$$R_F = \frac{b}{t}\sqrt{12\frac{\sigma_y}{E}\frac{1-\nu^2}{\kappa_F\pi^2}} \tag{2}$$

$$\gamma_l^* = \begin{cases} \frac{1}{n}\left[\{2n^2(1+n\delta_l)-1\}^2-1\right] & \left(\alpha \geq \sqrt{1+n\gamma_l}\right) \\ 4\alpha^2 n(1+n\delta_l)-\frac{(\alpha^2+1)^2}{n} & \left(\alpha < \sqrt{1+n\gamma_l}\right) \end{cases} \tag{3}$$

where b = width of flange; t = thickness of plate; E = Young's modulus; σ_y= yield stress ; ν = Poisson's ratio (=0.3) ; n = number of panels; k_R = bucking coefficient; α = aspect ratio; γ_l= relative stiffness ratio of an actual longitudinal stiffener.

Based on survey results shown in Fig.1, target values of major parameters of test specimen, R_R, R_F and γ_l/γ_l^* were selected as follows.

$$R_R = 0.5, \quad R_F = 0.5, \quad \gamma_l/\gamma_l^* < 1.0 \tag{4}$$

Steel grade of test specimens was SM570 in consideration of the steel grade of actual beam-to-column connections. The outside dimensions of specimens were made as large as possible in consideration of the capacity of the hydraulic jacks and frames.

It has been proposed that fillets are installed in beam-to-column connections in order to improve the fatigue performance. Fillets can reduce the high stress concentration at the

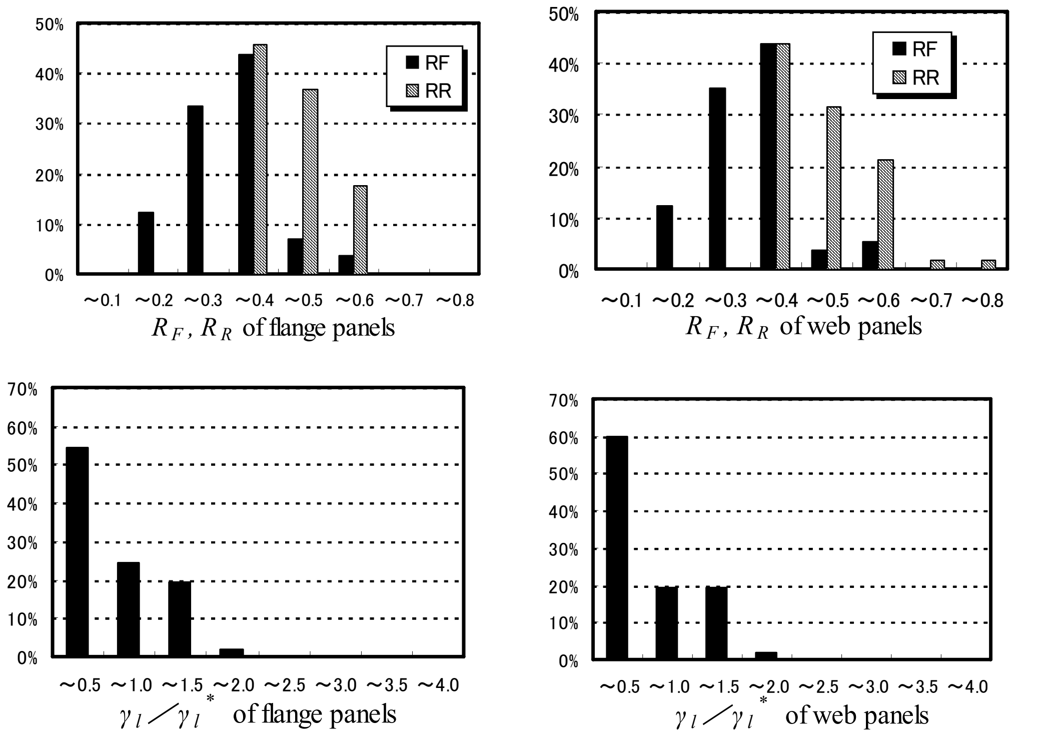
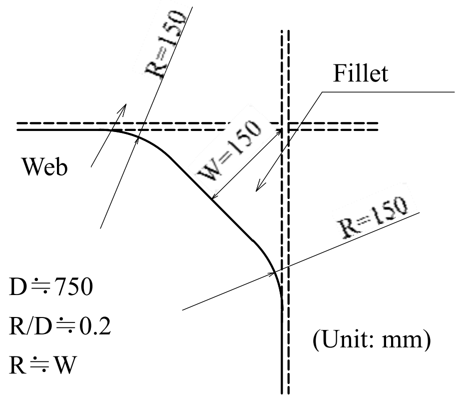


Fig.1 Survey results of R_R, R_F and γ_l / γ_l^* of actual beam-to-column connections



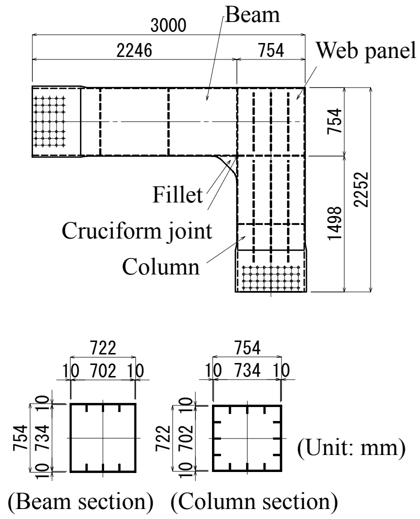


Fig.2 Geometry of fillets in test specimen

Fig.3 Geometry of test specimen C1

corners of beam-to-column connections. The geometry of the fillets has been already prescribed in design specifications for Metropolitan Expressway as follows[7].

$$W \fallingdotseq 0.2H_b \quad \text{and} \quad R \fallingdotseq \mathrm{W} \tag{5}$$

where W = width of fillet; R = radius of the end of fillet; H_b = height of web in column.

Geometry of fillets in specimens is shown in Fig.2.

Table 1 Major parameters of test specimens

	σ_{yM} (N/mm^2)	R_R	R_F	γ_l/γ_l*	γ_l/γ_{lreq}	δ_y (mm)
C1	562	0.48	0.52	0.85	1.38	8.8
C2	588	0.48	0.52	0.85	1.38	6.6

Note: R_R, R_F and δ_y was calculated by σ_{yM}

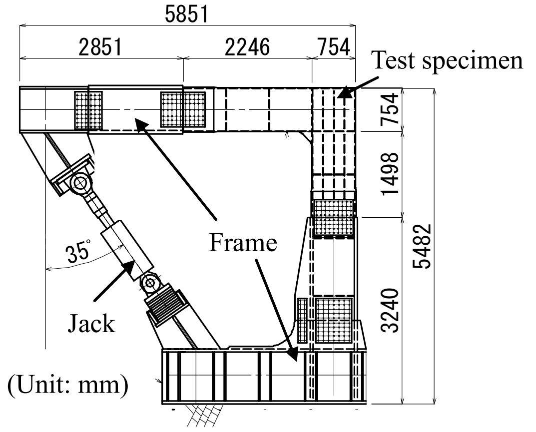


Fig.4 Outline of test setup

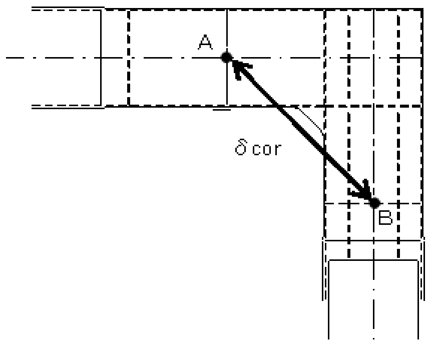


Fig.5 Displacement used for control of loading

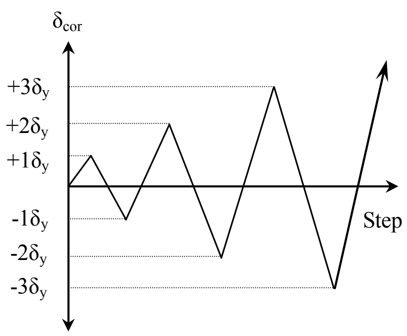


Fig.6 Cyclic loading program

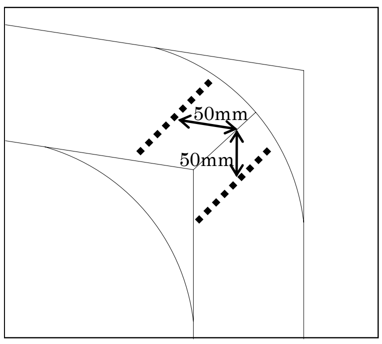
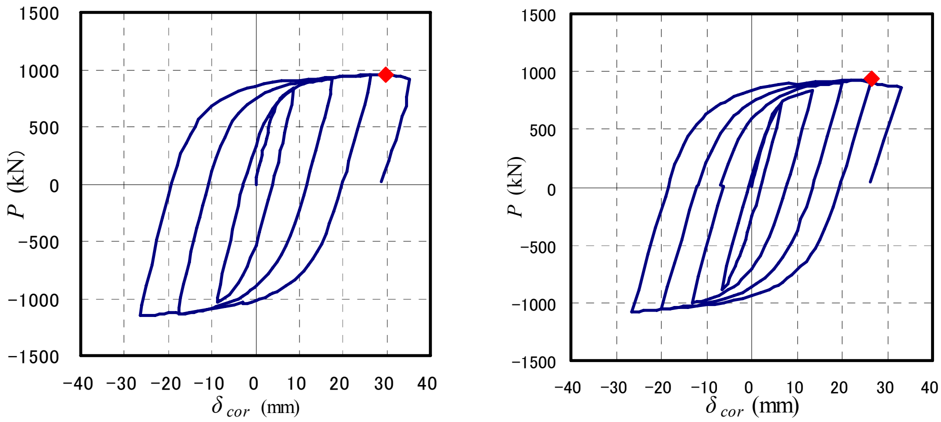


Fig.7 Distribution of Strain gauges for deciding δ_y

The geometry of a test specimen C1 with fillets is given in Fig.3 and the values of major parameters and material properties of the test specimens are listed in Table 1. Here, C1 is a test specimen with fillets and C2 is a test specimen without fillets. The geometry of a test specimen C2 is the same as that of C1 except fillets.

2.2 Cyclic Program

Fig.4 is an outline of test setup. Each specimen was set in fully stiff frames. A hydraulic jack was fixed at the end of each flame. The jack adapted an angle of about 35° to the vertical axis so that distribution of moment acting on beam-to-column connections of specimens and that of actual beam-to-column connections could be as approximate as



(Test specimen C1) (Test specimen C2)

Fig.8 *P*- δ_{cor} relationship

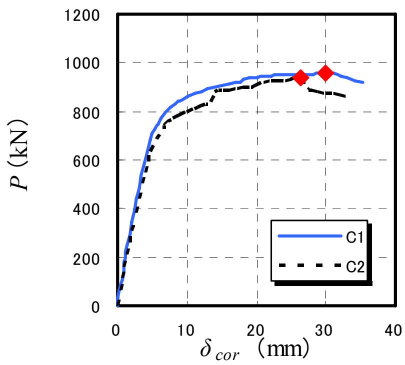


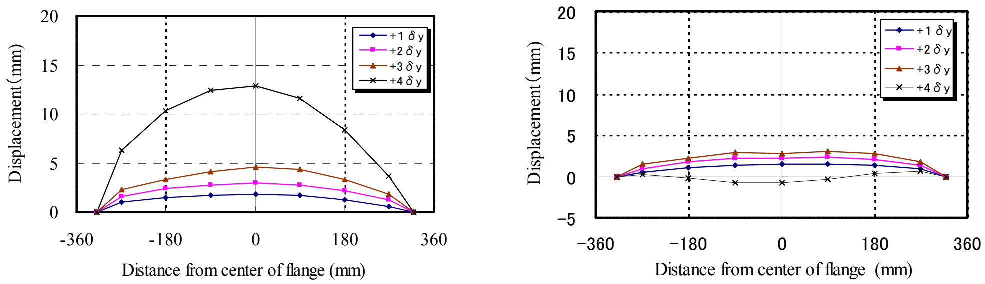
Fig.9 Envelop curves

possible.

Each experiment was controlled by the displacement δ_{cor}. δ_{cor} is a displacement as shown in Fig.5. The cyclic loading program is schematically shown in Fig.6. δ_y in Fig.6 is the yield horizontal displacement and it was decided by the following procedure. As shown Fig.7, strain gauges were attached on both surfaces of inner flange plates in the beam and the column and the distance between the gauges and the cruciform joint is 50 mm. The average values of all strain measured by strain gauges in the beam or the column were calculated respectively. The displacement δ_{cor} when the average value of the measured strain in the beam or the column reached to the yield strain ε_{ym} was defined as the yield displacement δ_y. Here, the yield strain ε_{ym} was obtained form the material test of each test specimen. The compressive force and the shrinking displacement from the neutral position were defined as the positive value respectively. By the way, yield displacement of a test specimen C1 is larger than that of a test specimen C2 in Table 1 because the average value of stress of C1 was smaller than that of C2 as the result of the fillets as described in the following section 3.3.

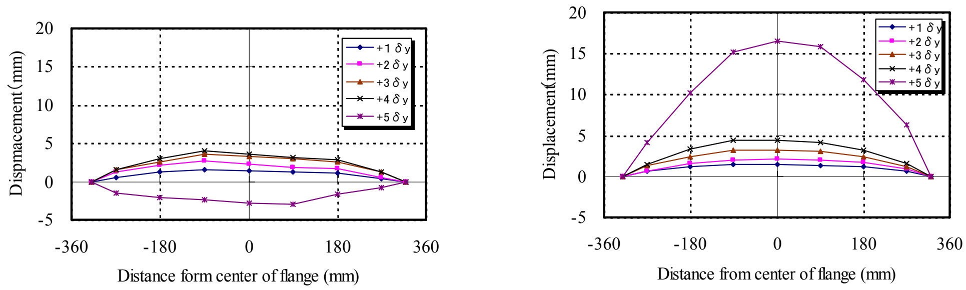
3 EXPERIMENTAL RESULTS AND COMMENTS

3.1 Feature of hysteretic curves



(a) Beam (b) Column

Fig.10 Progress of deformation on inner flange of C1



(a) Beam (b) Column

Fig.11 Progress of deformation on inner flange of C2

Fig.8 indicates a jack load P-displacement δ_{cor} relationship and Fig.9 shows the envelope curves of the test specimens. The square symbols (◆) in Fig.8 and Fig.9 exhibit the points where P_{max} appeared.

Fig.9 demonstrates that P_{max} and the displacement at P_{max} of C1 are a little larger than those of C2. However the envelop curve of C1 before P_{max} is almost identical with that of the specimen C2. Judging from the fact described above, fillets discussed in this paper have little effect on the overall load-displacement relationship of beam-to-column connections as far as the elasto-plastic behavior before P_{max} is concerned. On the other hand, difference between the envelop curve of C1 and that of C2 after P_{max} is larger than that before P_{max}. The reason why the larger difference in the envelop curves after P_{max} occurred will be investigated in detail hereafter.

3.2 Buckling behavior and low cycle fatigue performance

Fig.10 and Fig 11 show the progress of out-of-plane deformation of the inner flanges in the beam and the column at the end of each loop respectively. The points where the displacement was measured were 50 mm from the cruciform joint. And positive values of the deformation in Fig.10 and Fig.11 mean the out-of-plane displacement occurred into inside of flange plates.

As for a test specimen C1, the local buckling occurred with large deformation at the inner flange plates in the beam in the loading path of +4 δ_y as shown in Fig.12. This bucking caused decline of strength and P_{max} appeared. The deformation of the inner flange plate in the beam in the loop of +4 δ_y was much larger than that in the other loops as shown in Fig.10(a).

Regarding a test specimen C2, the cracks were found on the flange side surface of weld bead of the cruciform joint at the corner in the loop of -3 δ_y as shown Fig.13. These cracks propagated with the progress of the loading in the loop of -3 δ_y, but the load P didn't decline.

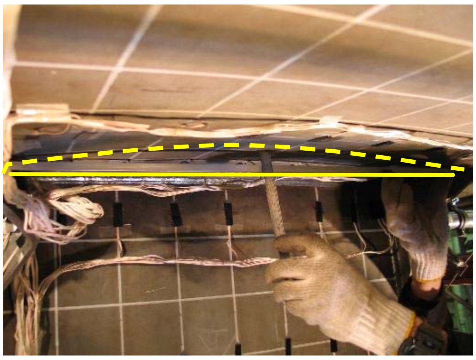


Fig.12 Local buckling at the inner flange in beam of C1 (+4 δ_y)

Fig.13 Cracks on flange side surface of weld bead in C2 (-3 δ_y)

Fig.14 Cracks propagation into web plate in C2(-4 δ_y)

Fig.15 Local buckling at inner flange in C2 in column of C2 (+5 δ_y)

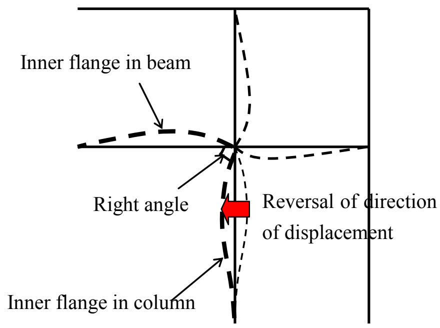


Fig.16 Process of appearance of reverse deformation

The cracks initially found in -3 δ_y propagated to web plates in the loop of -4 δ_y as shown in Fig.14. In the loading path of +5 δ_y , the local buckling occurred with large deformation at the inner flange plates in the column as shown in Fig.15 and this bucking caused decline of strength and appearance of P_{max} as well as C1.

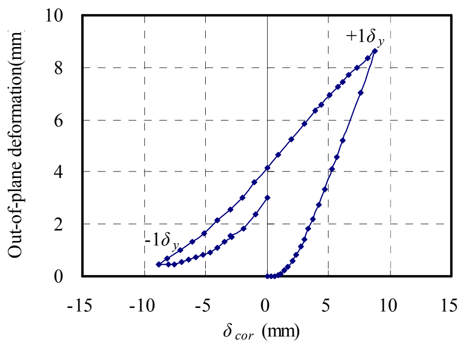


Fig.17 δ_{cor} - deformation of the fillets relationship

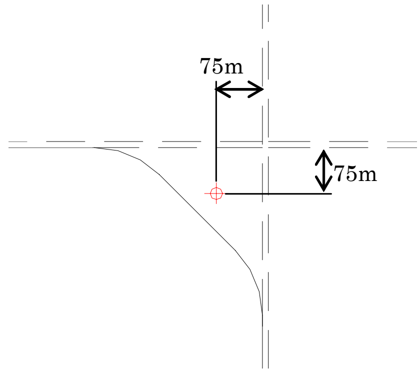


Fig.18 Measuring point of deformation

By the way, the deformation of the inner flange plate in the beam or the column was larger than that of another member and the direction of deformation in the beam was reverse to that in the column after P_{max}. As for a test specimen C1, the deformation of the inner flange in the beam was much larger than that in the column and the direction of deformation in the beam was reverse to that in the column in the loop of $+4\delta_y$ as shown in Fig.10. Moreover, Fig10 (b) indicates that the direction of the deformation of the inner flange in the column in the loops of $+4\delta_y$ was opposite to that of $+1\delta_y$, $+2\delta_y$ and $+3\delta_y$. The reason why this buckling behavior happened is supposed as hollows. The rigid of the part of the cruciform joint is much larger than that of flange plates because of the weld bead, so the right angle of the cruciform joint tends to be kept even if the large deformation occurs at the inner flange plates. Therefore, if the buckling occurs with large deformation at the inner flange plates in the column or the beam, the large deformation of one member may cause deformation of another member in the reverse direction for keeping the right angle of the cruciform joint as shown in Fig.16.

Fig.17 shows the relationship between δ_{cor} and out-of-plane deformation of the fillets in the loops of $+1\delta_y$ and $-1\delta_y$ at the point shown in Fig.18. In Fig.17, the positive values of the deformation mean the out-of-plane deformation occurred inside. Fig.17 indicates that the maximum value of the deformation in the loop of $+1\delta_y$ is about 9mm. And residual deformation remains and the value is about 3mm even though δ_{cor} returns to zero. The large residual deformation at fillets after the earthquake is thought not to be desirable. It is important to consider the residual deformation of fillets in developing the seismic design method of beam-to-column connections with fillets.

According to fact mentioned above, fillets have little effect on buckling behavior of the inner flange plate and can improve the low cycle fatigue performance of the cruciform joint under the earthquakes but it is important to consider the residual deformation of fillets.

3.3 Feature of stress distribution on inner flanges in elastic range

Fig.19 and Fig.20 show the stress distribution on the outside surface of inner flange in the beam at 50 mm and at 280 mm from the cruciform joint respectively. The point at 280 mm from the cruciform joint corresponds to the edge of the fillets. The stress in Fig.17 and 18 was in the elastic range and the load corresponding to the stress in these figures was about 300kN in both C1 and C2.

Fig.19 demonstrates that stress of C1 is smaller than that of C2 and especially the high stress concentration at the corners of C1 is much smaller than that of C2 at 50mm from the

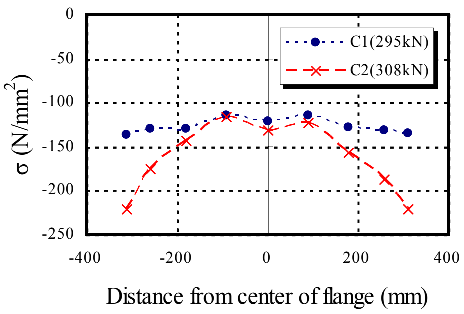


Fig.19 Stress distribution in beam at 50mm from cruciform joint

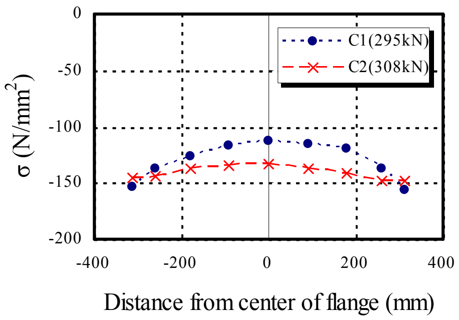


Fig.20 Stress distribution in beam at 280mm from cruciform joint

cruciform joint. On the other hand, Fig.20 indicates that the stress of C1 is overall smaller than that of C2 but the stress concentration at the corners is a little larger than that of C2 at 280mm from the cruciform joint as the result of the edge of the fillets.

So fillets can reduce the stress near the cruciform joint overall. Especially, they can decrease stress concentration at the corners greatly. However they may increase the stress concentration at the edge of fillets.

4 CONCLUSION

Cyclic loading experiments of beam-to-column connections of steel bridge frame piers were carried out in order to grasp of the elasto-plastic behavior of beam-to-column connections with fillets and the influence of fillets on it. Based on the experimental results, the following conclusion can be drawn.

1) Fillets dealt with in this paper have little effect on the overall load – displacement relationship of beam-to-column connections before P_{max} and on buckling behavior of the inner flange plates of beam-to-column connections. However it is important to consider the residual deformation of fillets.
2) Fillets can improve the low cycle fatigue performance of the cruciform joint.
3) Fillets can reduce the stress near the cruciform joint overall. Especially, they can decrease stress concentration at the corners greatly. However fillets may increase the stress concentration at the edge of fillets.

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