Comparison and Analysis of Design Procedures for CFST-to-Steel Girder Connection Panel Zone Shear

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SUMMARY:
In this work, experiments on composite connections consisting of steel girders framing into concrete-filled steel tube (CFST) beam-columns conducted in recent years were investigated and the test results were compared with the calculation results of the AIJ specification, the calculation method proposed by Fukumoto and Morita (2005), and the calculation method verified by Nishiyama et al. (2004). Through data analysis, the study confirmed the applicability of each of the three methods. The ultimate shear strength calculation results of Fukumoto and Nishiyama were found to be more accurate and reliable than AIJ specification procedure, and the shear yield strength calculation results of the two methods were also shown to be appropriate. Through a parametric analysis, this study confirmed that the three methods have a broad applicability for joints of various axial compression ratios, connection formations and column cross sections. The analysis results suggested that the applicable steel tensile strength and concrete compression strength for the AIJ specification should not exceed 450 MPa and 70 MPa, respectively, and the applicable ratio of steel tensile yield strength to concrete compression strength should not exceed 8 for the AIJ procedure to yield accurate results. This paper also provides proposed amendments to these procesures for joints having special connection details.

Keywords: concrete-filled steel tube; steel girder; panel zone; composite connection

1. INTRODUCTION

Concrete-filled steel tube (CFST) beam-columns combine key advantages of both steel members and concrete members, In building structures, CFSTs should have reliable connections with steel girder and floor slabs so as to comprise the lateral seismic load resisting system for the structure. Therefore, the calculation method and fabrication procedures for CFST composite connections are important provisions to ensure accurate and safe structural systems. In recent years, the connections of composite floor with profiled steel sheeting to CFST columns has become more prevalent, and several experiments have been conducted on steel girder-to-CFST connections by various researchers worldwide (e.g.,Ricles et al. (1995, 2004), Elremaily and Azizinamini (2001), Fukumoto and Morita (2005)). Typical joints of steel girders to CFSTs are shown in Figure 1. However, most of the tested joints failed at the end of the beam by flexure or within the connection components, and rarely has the nonlinear response of the panel zone been explored in detail. To ensure a proper failure progression and ductility of these structures, the panel zone should typically be stronger than the neighboring girders and columns. However, few calculation procedures for panel zone shear strength have been proposed.

Previous research on the panel zone of CFST-steel girder joints includes a series of experimental tests and some theoretical models. Moreover, some calculation methods derived from respective theoretical models have been proposed as well. Research involving experimental studies and theoretical models includes that of AIJ(1987), Fukumoto et al. (2000), Fukumoto et al. (2005), Cheng et al. (2003), Cheng et al. (2007), and Zhang et al. (2001). Models and calculations proposed by AIJ(1987), Fukumoto et al. (2000), Fukumoto et al. (2005) are all based on superposition of steel tube and core concrete, and they are laconic and may have applicable promise. However, influence of axial compression ratio is not taken into account and the applicable concrete standard compression strength and steel yield strength is limited to 36MPa and 490MPa respectively in AIJ (1987). Models by Fukumoto et al. (2000) and Fukumoto et al. (2005) are based on compression truss model and can take the confinement of steel tube to concrete and the influence of axial compression ratio into account. The difference between the two models is the distribution of axial load between steel tube and core concrete during nonlinear behavior. Models by Cheng et al. (2003) and Cheng et al. (2007) are based on compression truss model and can take axial compression ratio into account as well; however, it is too complex to evaluate the shear capacity of core concrete. Zhang et al. (2001) proposed a model of failure face and derived calculation formula of panel zone shear capacity by numerical integration on the failure face. By regression of the test data, Koester (2000) proposed a formula for joints having through-bolted and split-tee details, yet the absence of theoretical model make its applicability limited. Experimental research includes that of Nishiyama et al. (2004), Elremianly et al. (2001), Ricles et al. (1995), Ricles et al (2004), Wu et al. (2005), Takemura et al. (1999), Han (2009) and Kamba et al. (1991). Moreover, Nishiyama et al. (2004) also confirmed that the applicable concrete standard compression strength and steel yield strength of AIJ (1987) is up to 110MPa and 809MPa respectively and the model proposed by Fukumoto et al. (2000) coincided well with test results.

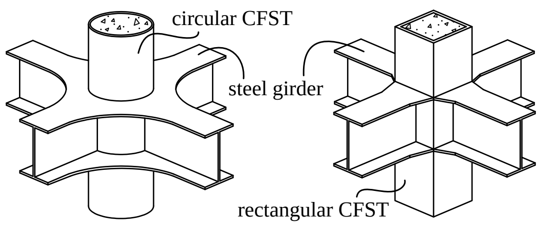

Figure 1. Typical CFST-H shaped steel girder joint

This research makes use of the above models and experimental results of CFST-to-steel girder composite joint panel zones and selects three applicable calculation methods for further research. Parametric analysis is conducted to analyze the features and applicable range of different parameters for the three methods. The research results can provide reference for seismic design of CFST-to-steel girder joints.

2. SHEAR STRENGTH CALCULATION METHODS FOR COMPOSITE PANEL ZONES

This section summarizes three methods for computing the panel zone shear strength of composite CFT connections.

2.1. Calculation Method in AIJ (1987)

The AIJ specification (AIJ, 1987) provides a method for evaluating panel zone shear strength of CFST-to-steel girder joints, as shown in Eq. (2.1). The first term of Eq. (2.1) is associated with the contribution of the core concrete to the panel zone shear capacity, while the second is associated with the contribution of the steel tube. In Eq. (2.1), A_c, d_{sb}, f_c, A_{web}, D, f_y, $\beta_{J,CFT}$ and $\beta_{J,RCFT}$ are the cross-sectional area of concrete, the distance between the centroids of the beam flanges, the concrete compression strength (in MPa), the area of the web of the steel tube (if the steel tube is circular, this variable equals half the area of the tube cross section), the cross-sectional height of the steel tube web, the yield tensile strength of the web of steel tube (in MPa), the coefficient for circular column cross sections and the coefficient for rectangular column cross sections, respectively.

$$\begin{cases} V_{pu} = V_c F_{JS} \beta_J + 1.2 V_s f_y / \sqrt{3} \\ V_c = A_c d_{sb} \\ V_s = A_{web} d_{sb} \\ F_{JS} = \min(0.12 f_c, 1.8 + 3.6 f_c / 100) \\ \beta_{J,CFT} = \min(2D / d_{sb}, 4) \\ \beta_{J,RCFT} = \min(2.5D / d_{sb}, 4) \end{cases} \tag{2.1}$$

2.2. Calculation Method proposed by Fukumoto and Morita (2005)

Fukumoto's method for computing composite panel zone strength (Fukumoto and Morita, 2005) is derived from a constrained compression truss model, obtaining the panel zone shear strength by directly superimposing the contribution of the steel tube and core concrete. The calculation is shown in Eq. (2.2), where β is the ratio of the shear yield load to shear ultimate load of the core concrete, and β_{CFT} and β_{RCFT} are the coefficient β for circular column cross sections and rectangular column cross sections, respectively. The terms h, D, f_c, f_y, A_w, b_c, t_{cf} are the cross-sectional depth of the beam, the cross-sectional height of the steel tube web, the concrete compression strength (in MPa), the tensile yield strength of steel tube (in MPa), the area of the web of steel tube (if the steel tube is circular, this variable equals half the area of the tube cross section), the cross-sectional width of the steel tube flange, and the thickness of the flange of the steel tube. N/N_0 is the axial compression ratio of column, where $N_0 = A_s f_y + A_c f_{ck}$.

$$\begin{cases} V_{py} = V_{sy} + \beta V_{cu} \\ V_{pu} = V_{sy} + V_{cu} \\ \beta_{CFT} = 0.228 h / D + 0.520 N / N_0 + 0.295 \\ \beta_{RCFT} = 0.425 N / N_0 - 1.13 f_c / f_y + 0.650 \\ V_{sy} = A_w \sqrt{f_y^2 - \sigma_{SN}^2} / \sqrt{3} \\ \sigma_{SN} = N A_s f_y \big/ [A_s (A_s f_y + A_c f_c)] \\ V_{cu} = (D \tan\theta / 2 + 4 \sqrt{M_{fp}^2 / D f_c} \sin\theta) D f_c \\ \theta = \tan^{-1}(\sqrt{1 + (h/D)^2} - h/D) \\ M_{fp} = b_c t_{cf}^2 f_y / 4 \end{cases} \tag{2.2}$$

2.3. Calculation Method Verified by Nishiyama et al. (2004)

The method outlined and verified by Nishiyama et al. (Nishiyama et al., 2004) is similar to the method proposed by Fukumoto and Morita (2005). The primary difference is embodied within the parameter σ_{SN}, as shown in Eq. (2.3). The distribution of axial load is determined by the material strength in Fukumoto's method, while it is determined by elastic rigidities in the method verified by Nishiyama. In Eq. (2.3) E_s, E_c are the elastic modulus of the steel and concrete, respectively.

$$\sigma_{SN} = NA_s E_s / [A_s (A_s E_s + A_c E_c)] \tag{2.3}$$

3. COMPARISON AND PARAMETRIC ANALYSIS OF THE METHODS

In order to evaluate the accuracy and reliability of the methods, they are verified by a subset of the composite joint conducted between 1991 and 2008, which include 24 cyclic tests and 20 monotonic tests. The applicable range of material parameters and geometric parameters of the methods are also proposed. Based on the referred test data, the applicable range of parameters such as axial compression ratio, joint type, joint connection detail and other material or geometric parameters are analyzed. The analysis results are the ratios of tested shear yield load V_y to calculated shear yield load V_{py} and tested shear ultimate load V_u to calculated shear ultimate load V_{pu}, respectively. The experimental yield load is defined as the corresponding load at the point that has the maximum curvature in the load-displacement curve.

3.1. Parametric Analysis of Model Details

The relationship of axial compression ratio (N/N_0) and the ratio of cyclic tested load to calculated load (V_y/V_{py}, V_u/V_{pu}) is shown in Figure 2, and the relationship of axial compression ratio (N/N_0) and the ratio of monotonic tested load to calculated load (V_y/V_{py}, V_u/V_{pu}) is shown in Figure 3. Excluding the specimen that did not develop full capacity because of beam failure, column compression-bending failure or connection fracture failure, most of the calculated ultimate load is safe for engineering application. However, the calculated ultimate load is clearly a bit conservative when axial compression ratio is between 0.2 and 0.3, and the results of Fukumoto method and Nishiyama method are more accurate than that of AIJ specification in this condition as well. The yield load may seem a bit unsafe in some cases, but it is still applicable for engineering design if revised with a safety coefficient.

The relationship of column cross section type and the ratio of tested load to calculated load (V_y/V_{py}, V_u/V_{pu}) is shown in Figure 4. It can be seen that the ultimate load of AIJ method is less accurate than that of Fukumoto method and Nishiyama method if column cross section is circular, and the ultimate loads of Fukumoto method and Nishiyama method have almost the same accuracy for both column cross sections. The yield load of circular column cross section is less accurate or reliable than that of rectangular column cross section since discreteness of circular column cross section is a bit more obvious.

The relationship of connection detail and the ratio of tested load to calculated load (V_y/V_{py}, V_u/V_{pu}) is shown in Figure 4. Although the specimen which had connection detail of external ring and were tested by Han et al. did not develop full capacity because of insufficient column compression-bending capacity, it is still possible that the calculated ultimate load of joints with connection detail of external ring by Fukumoto method and Nishiyama method maybe unsafe. However, AIJ specification is relatively safe for joints with connection detail of external ring. Excluding joints with connection detail of external ring, the ultimate load by three methods are all safe according to the analysis. The ultimate load of joint with connection detail of through diaphragm is a bit discrete by all methods and some specimens are too conservative. The ultimate load of joint with connection detail of through beam is a bit conservative as the contribution of beam web in panel zone could not be considered.

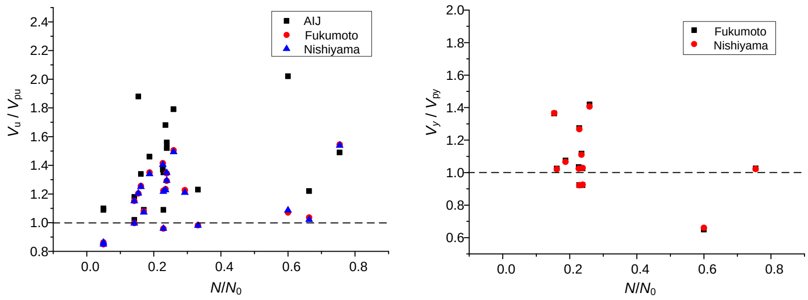

Figure 2. Relationship of N/N_0-V_u/V_{pu} and N/N_0-V_y/V_{py} under cyclic loading

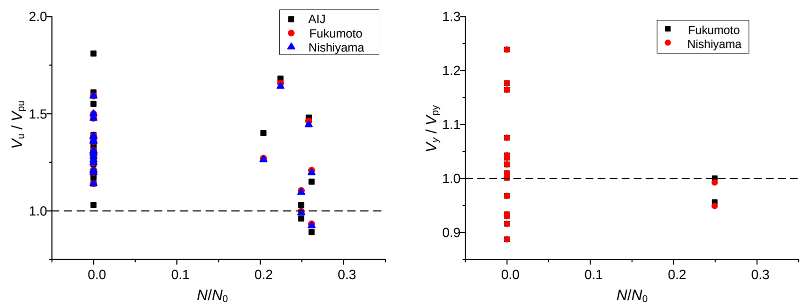

Figure 3. Relationship of N/N_0-V_u/V_{pu} and N/N_0-V_y/V_{py} under monotonic loading

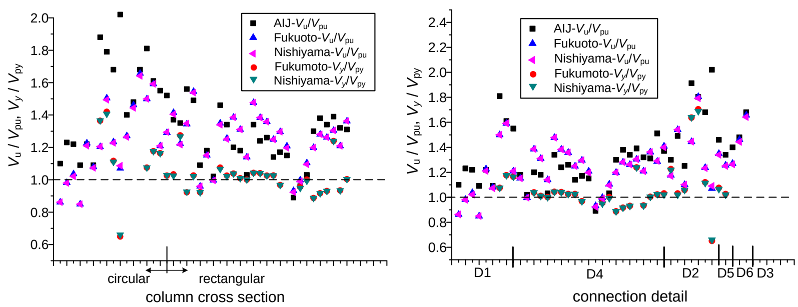

Figure 4. Relationship of column cross section, connection detail and V_u/V_{pu} , V_y/V_{py}

The specimen whose yield load is clearly unsafe in Figure 2 is an exterior joint. Moreover, the ultimate loads of three methods present significantly discrete results as well. As the number of exterior joint specimen is too small, further research is needed. Since some parameters of Fukumoto method and Nishiyama method are obtained by regressing test data of interior joints, it still need to be confirmed whether the methods are applicable for exterior joints.

3.2. Parametric Analysis of Material Strength

The relationship of concrete strength f_c and the ratio of tested load to calculated load (V_y/V_{py}, V_u/V_{pu}) is shown in Figure 5. Excluding the specimen that did not develop full ultimate capacity for other reasons, ultimate loads of the three methods are all safe when f_c does not exceed 110MPa. However, ultimate load of AIJ specification disperses significantly when f_c exceeds 70MPa and it is proposed that the applicable range of f_c had better not exceed 70MPa. Yield load is all satisfactory and can meet the demand of engineering design when f_c does not exceed 110MPa.

The relationship of steel strength f_y and the ratio of tested load to calculated load (V_y/V_{py}, V_u/V_{pu}) is shown in Figure 6. Ultimate load of the three methods is all safe when f_y does not exceed 900MPa. However, the ultimate load of AIJ specification disperses significantly when f_y exceeds 450MPa and it is proposed that the applicable range of f_y had better not exceed 450MPa. It is obvious that the calculated yield load maybe a bit unsafe when f_y does not exceed 450MPa, yet it is reasonable since some parameters in the methods are regressed from tests fabricated with high strength steel. However, it does not prevent the method from being applied in engineering design.

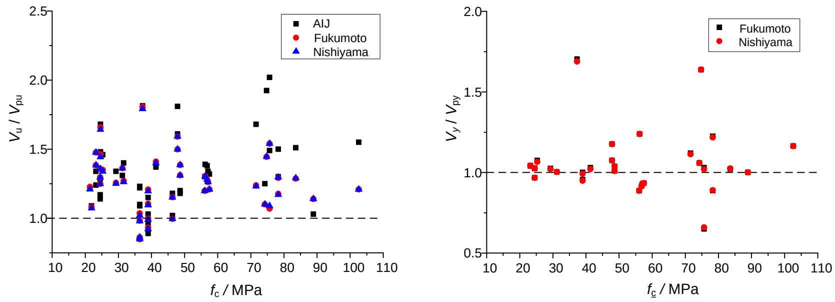

Figure 5. Relationship of f_c-V_u/V_{pu} and f_c-V_y/V_{py}

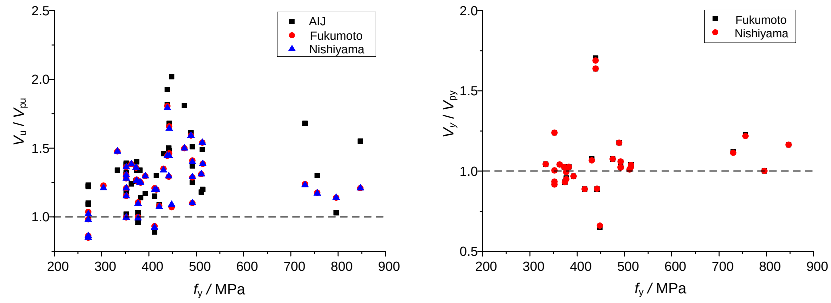

Figure 6. Relationship of f_y-V_u/V_{pu} and f_y-V_y/V_{py}

The relationship of material strength ratio (f_y/f_c) and the ratio of tested load to calculated load (V_y/V_{py}, V_u/V_{pu}) is shown in Figure 7. The ratio of f_y to f_c has little influence on ultimate load while has obvious effect on yield load. The calculated yield load becomes unsafe when the ratio of f_y to f_c is below 8. Further research confirms that this phenomenon does not be affected by numerical value of f_y or f_c, so it is proposed that the ratio of f_y to f_c had better exceed 8. It is also found that the specimen whose yield load is unsafe all have a rectangular column cross section and the yield loads concentrate between 0.9 and 1.0. Based on the assumption of deformation compatibility, the yield strain of panel zone is selected as the shear yield strain of steel tube web, thus the shear strain of concrete is between cracking strain and ultimate strain at this point. Since the parameter β that represents the ratio of panel zone yield load to ultimate load is derived from test data regression and does not have a clear physical implication, it is proposed that the parameter β can be revised with a constant coefficient 0.9 when the ratio of f_y to f_c does not exceed 8. The revised parameter is shown as $\beta^{'}=0.9\beta$ and the contribution of core concrete to yield load is $\beta^{'}V_{cu}$.

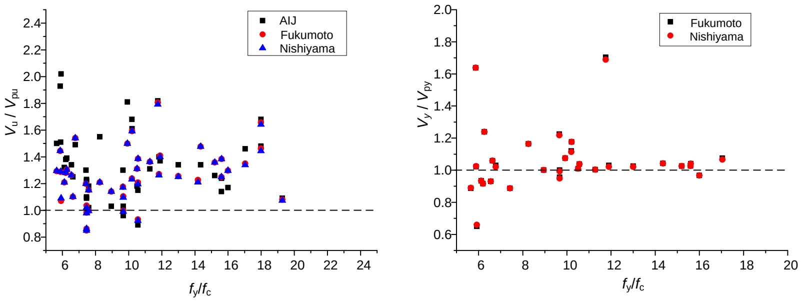

Figure 7. Relationship of f_y/f_c-V_u/V_{pu} and f_y/f_c-V_y/V_{py}

3.3. Parametric Analysis of Geometrical Dimension

The height-thickness ratio (D/t) of rectangular tube web has almost no influence on either ultimate capacity or yield capacity while the influence of circular tube diameter-thickness ratio on yield capacity is not significant as well. The relationship of circular tube diameter-thickness ratio (D/t) and ultimate load results (V_u/V_{pu}) is shown in Figure 8. The ratio of tested capacity to calculated capacity has a decreasing tendency with the increase of diameter-thickness ratio (D/t).

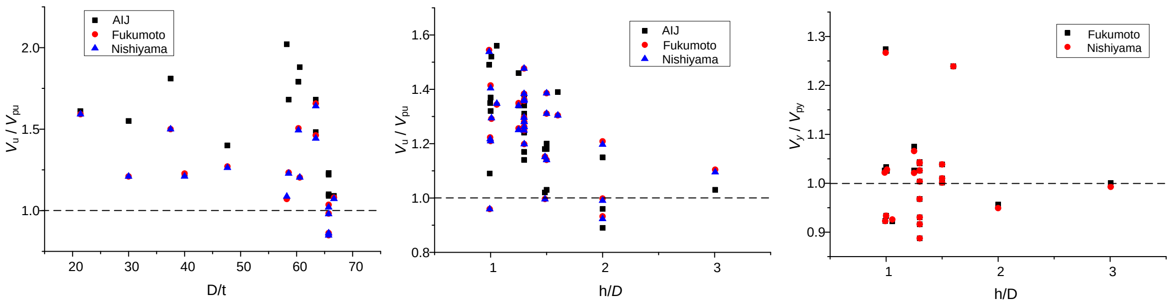

Figure 8. Relationship of D/t-V_u/V_{pu}, h/D-V_u/V_{pu} and h/D-V_y/V_{py}

Based on data analysis, it is confirmed that the panel zone aspect ratio (h/D) has no significant influence on calculation results of joints with circular columns. The relationship of rectangular panel zone aspect ratio (h/D) and the ratio of tested load to calculated load (V_y/V_{py}, V_u/V_{pu}) is shown in Fig. 8. With increase of aspect ratio, ultimate load goes unsafe significantly. However, three methods are all safe if aspect ratio does not exceed 1.5 while panel zone aspect ratios of actual engineering structures are all within this value. Therefore, they are all applicable for actual engineering structures.

4. PROPOSED AMENDMENTS FOR THE METHODS

4.1. Panel Zone Height Redefinition of Joints Having Through-bolted End Plate Detail

In typical joints, the panel zone height is defined as the distance between the tensile flange and the compressive flange of the adjoining girder. It is often defined as the distance between both centroids of the distributed forces on tensile and compressive flanges. In joints that have a connection detail consisting of a through-bolted end plate, the tensile force of the girder flange is transferred to the panel zone by the bolts, making the mechanism of joints having a through-bolted end plate detail different with other connection types. Therefore, the centroid of the tensile forces should be redefined as the centroid of the bolt tensile forces. The panel zone height can be redefined as the distance between the centroid of the bolt tensile forces and the centroid of the girder compressive flange. If the end plate has sufficient stiffness and the bolts are symmetrically distributed around the tensile flange, the panel zone height may be simply defined as the distance between the tensile flange and compressive flange as well.

4.2. Formula Revision of Joints Having Through Beam Detail

The girder web has a significant contribution to shear strength of the joints having a detail of a through beam or through web, so the calculation method should be revised to take into account the contribution of the girder web. The contribution of the girder web is defined as V_{sb}, as shown in Eq. (4.1), and it should be directly superimposed within the calculation of the shear yield load and the shear ultimate load of the panel zone. A_w and f_{ybt} are the area and the tensile yield strength of girder web, respectively:

$$V_{sb} = A_{wb} f_{ybt} / \sqrt{3} \tag{4.1}$$

The calculation results of the revised formula are shown as Table 1. The calculation results of composite joints numbered NSF1 and NSF7 show a large decrease, and the contribution of the girder web is significant. Specimen NSF1 did not develop its full strength because of a weld fracture failure. The calculation results of the revised formula is more accurate on these specimens.

Table 1. Calculation results of revised formulas

Specimen number	AIJ- V_u/V_{pu}		Fukumoto- V_u/V_{pu}		Nishiyama- V_u/V_{pu}	
	with V_{sb}	without V_{sb}	with V_{sb}	without V_{sb}	with V_{sb}	without V_{sb}
NSF1	0.88	1.40	0.83	1.27	0.82	1.26
NSF7	1.36	1.68	1.35	1.66	1.34	1.64

5. CONCLUSIONS

Three calculation methods for panel zone shear strength of composite connections consisting of steel girders framing into concrete-filled steel tubes were verified with test data. Through parametric analysis, the methods were compared and the applicable ranges of various parameters were proposed. Key conclusions include:

(1). The Fukumoto method, Nishiyama method and AIJ specification approach for evaluating CFST-to-steel girder composite joint panel zone shear strength are all applicable for design; the Fukumoto method and Nishiyama method are typically seen to be more accurate and reliable than the AIJ specification approach.

(2). Each method has a wide applicable range for parameters such as the axial compression ratio, connection detail and various geometric or material parameters. The three methods are all distinctly conservative when the axial compression ratio in the column is in the range of 0.2 to 0.3. However, further research is still needed to develop procedures for accurate assessment of the panel zone shear capacity strength for exterior joints.

(3). the AIJ specification is less accurate when the concrete compression strength f_c exceeds 70 MPa or the steel tensile yield strength f_y exceeds 450 MPa, or when the material strength ratio of the steel tensile yield strength to the concrete compression strength (f_y/f_c) exceeds 8.

(4). The calculation procedures for panel zone strength should be revised to account for the actual force delivery mechanisms when applied to joints having connection details such as through-bolts, whereby the force is transferred at the bolt line rather than at the girder flange centroid.

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