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NUMERICAL STUDY ON SHEAR PANEL BEHAVIOR OF PANEL ZONE IN WEB-CLAMPED TYPE CONNECTION

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Abstract

The web-clamped type connection is proposed as a new connection which has stiffness comparable to conventional welded connections and high workability comparable to conventional high strength bolted connections. In this connection, the prefabricated attachments are connected to the column web directly, two slits are opened on the column flange. This connection detail makes the force from beam transfer to the column web directly.

Two main features of web-clamped type connection which may cause the unique panel zone behavior are: 1) there is no horizontal stiffener in the panel zone, and 2) it is not necessary to connect the beam web and the column by the shear plate in the web-clamped type connection. In this connection, the attachment clamps a part of the panel zone, which can transfer the force from beam to column web directly. Owing to this unique detail, as shown in previous numerical studies, the attachments can transfer both shear force and bending moment from beam, which leads to an omission of the shear plate.

Because of this two features of the panel zone in the web-clamped connection, the boundary condition of the panel zone in the web clamped type is different from that of the conventional welded one, which might cause different panel zone behavior. In this paper, in order to investigate whether the panel zone of the web-clamped type connection show both a shear panel behavior in the elastic range and a truss behavior in the inelastic range, which is the same as that of the conventional welded connection, parametric numerical simulations are conducted and the strength, the stiffness and the stress distribution of the panel zone are compared.

As a result, when the strength and the stiffness of the panel zone was compared through the shear deformation angle of the panel zone between web-clamped models and conventional welded models, it was observed that the strength and stiffness of the panel zone of the web-clamped type models were 1.1 times higher than those of the conventional welded type models.

When the shear stress distribution was compared, the shear stress acting over the panel zone in the conventional welded connection was uniform, while that of the web-clamped connection was not uniform and high at the center of the panel zone. This might imply that the panel zone of the web-clamped type connection has another shear force transmission mechanism in addition to the shear panel behavior.

When deformation shape of the panel zone in the large panel shear deformation angle was compared, all the models showed the diagonal panel shear buckling and the diagonal tension field. This may imply that the internal force transfer mechanism of the panel zone of web-clamped connection switches to truss behavior after the panel zone yields, which is the same behavior as that of the conventional welded connection. Especially in the web-clamped type models without the shear plate, deflection in the vertical side of the panel zone was observed, which may cause the early strength deterioration of the panel zone.

Keywords: Beam-to-column Connection; Panel Zone; FE Analysis;

1. Introducion

The web-clamped type connection was proposed as a new connection which has stiffness comparable and high workability comparable to conventional high strength bolted connections. The configuration is shown in Fig. 1. In this connection, the prefabricated attachments connect the column web and beam flange by super high strength bolts. In order to connect the attachment and column web directly, two slits are opened on the column flange. This connection configuration makes the force from beam transfer to the column web directly.

Fig. 2 shows the comparison of the panel zone of the web-clamped type connection with that of the conventional welded type connection. Two main features of web-clamped type connection which may cause the unique panel zone behavior are: 1) there is no horizontal stiffener in the panel zone, and 2) it is not necessary to connect the beam web and the column by the shear plate in the web-clamped type connection. In this connection, the attachment clamps a part of the panel zone, which can transfer the force from beam to column web directly. Owing to this unique detail, as shown in the previous studies, the attachments can transfer both shear force and bending moment from beam [1,3], which leads to an omission of the shear plate.

Because of this two features of the panel zone in the web-clamped connection, the constraint effect of the panel zone in the web clamped type is different from that of the conventional welded one, which might cause different panel zone behavior. In this paper, in order to investigate whether the panel zone of the web-clamped type connection show a shear panel behavior and a truss behavior, which is the same behavior as that of the conventional welded connection, parametric numerical simulations were conducted and the strength, the stiffness, the stress distribution and the deformed shape of the panel zone were compared.

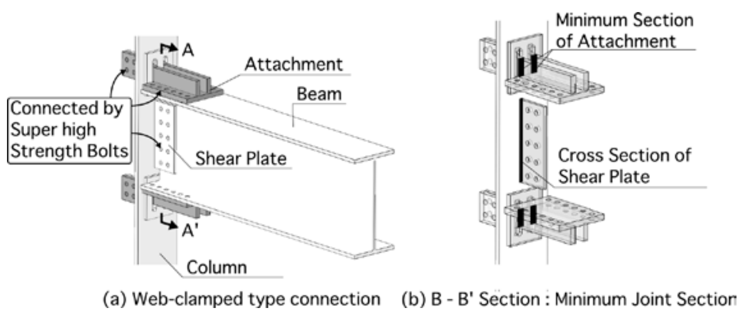


Fig. 1 – Overview of Web-clamped Type Connection

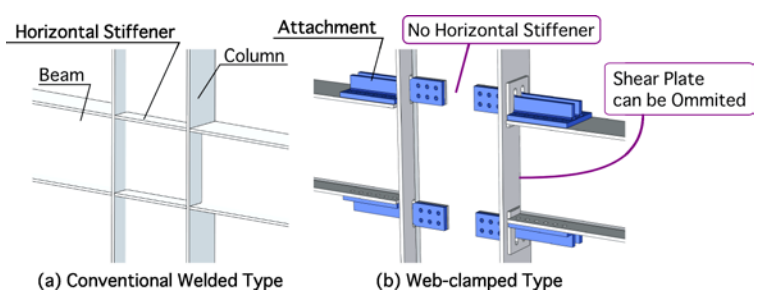


Fig. 2 –Comparison of the Panel Zone

2. Development of Finite Element Models

2.1 Analysis Model Geometry

Table 1 shows the main specifications of the numerical models used in this paper. In the study, the parametric study was carried out by varying the connection type (whether web-clamped type connection or conventional welded type connection), the axial ratio of the column and the existence of the shear plate. WSP series represent models of web-clamped connection in the case that beam web and the column are connected by the shear plate, WNSP series represent models of web-clamped connection in the case that the shear plate is omitted, and CONV series represent models of conventional welded connection. Fig. 3 shows the geometry of each model. The connection was modeled in full-scale size. In CONV series, the beam height is different from WSP and WNSP series to make the same lever arm of panel shear force.

Table 1 – Main Specification of the models conducted in this paper

No.	Name	Connection Type	Beam Web and Column Flange	Axial Force Ratio n	Column Section	Beam Section
1	WSP00	Web-Clamped Type	Bolted through Shear Plate	0.0	H-500x300 x12x22	H-600x300x 12x19
2	WSP04			0.4		
3	WNSP00		None	0.0		
4	WNSP04			0.4		
5	CONV00	Conventional Welded Type	Welded	0.0		H-744x300x 12x19
6	CONV04			0.4		

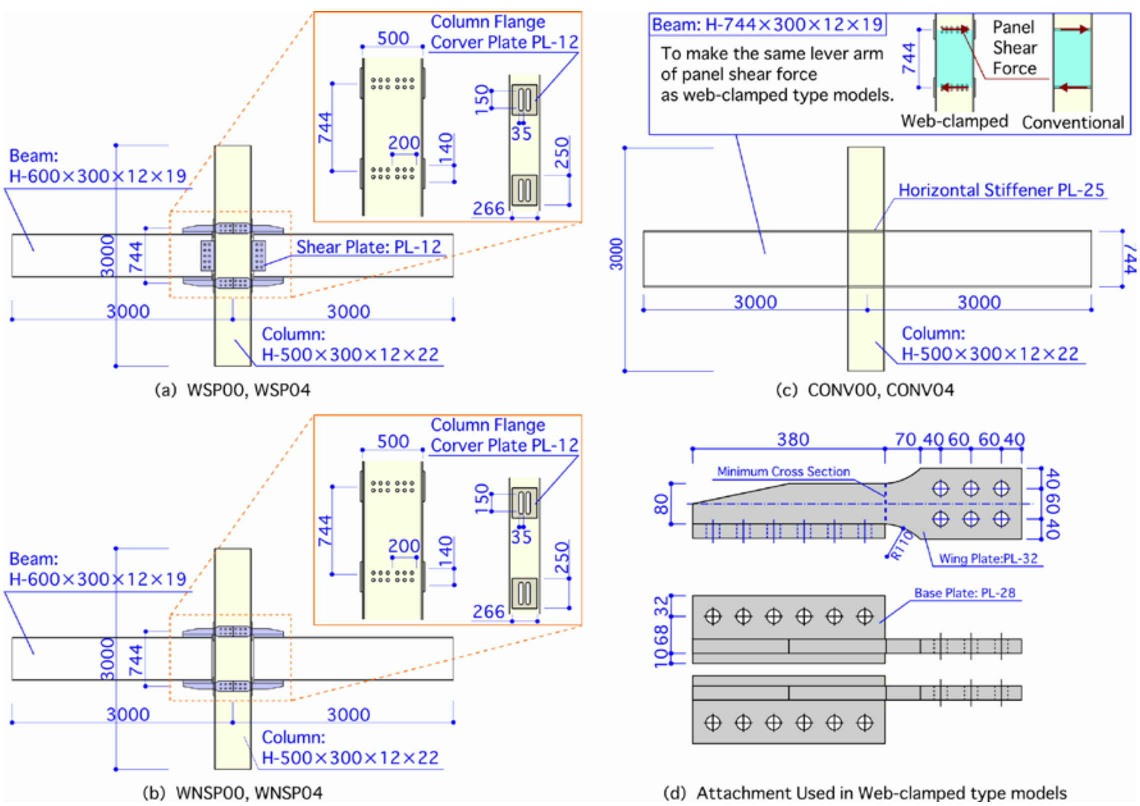
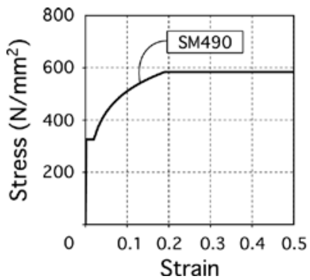


Fig. 3 – Shape of Models Conducted in This Paper

2.2 Material Properties

The mechanical properties of the steel used in the study and the corresponding stress strain curves are summarized in Fig. 4. This material used for all the parts of the models and the grade of steel is JIS SM490, which is equivalent to ASTM A572. The shape of the curve was derived from the tensile coupon test results of the material[4].



Steel	Young's Modulus (N/mm^2)	Yield Strength (N/mm^2)	Tensile Strength (N/mm^2)
SM490	2.05×10^5	325	490

Fig. 4 – Stress-strain curve and mechanical properties of the material

2.3 FE Analysis Modeling

Fig. 5 shows the summary of finite element model and Fig. 6 shows the definition of displacement and forces acting on the model. As shown in Fig. 5, a vertical deformation was statically applied to the beam end, so that the connection should be subject to anti-symmetric bending moment. The model is created with shell elements and analyzed with ABAQUS ver 6.12[5]. The mesh size of the panel zone, attachments, beam and column is 10mm.

Displacement-control analysis was conducted by imposing vertical displacements on the top of the beam until the connection rotation angle θ (see Fig. 6) was 0.1 rad. Column axial force was applied for the models of axial force ratio n=0.4 for the models of WSP04, WNSP04 and CONV04. In the bolted connection, relative displacement and rotation were constrained at the contact surface, so that bolt slip in the bolted connection is not simulated in the analysis, as shown in Fig.5.

To simulate shear buckling of the panel zone, initial imperfections, which were obtained by eigenvalue analysis, were introduced before static pushover analysis. The initial imperfection used for the analysis was the buckling mode corresponds to the shear buckling in the panel zone and introduced so that the maximum deformation of the buckling mode was 0.1mm.

2.4 Estimated Strength

Estimated strength of the connection is also shown in Table 2. All the models were designed so that the panel zone yielded first and the ultimate strength of the connection was determined by the full plastic moment of the panel zone.

In this table, all the estimated strength is shown as the panel moment ${}_pM$ (see Fig. 6), which is defined as

$${}_pM = {}_pQ \cdot d_k \tag{1}$$

$${}_pQ = 2P \cdot l_{b2} / d_k - R \tag{2}$$

$$P \cdot l_b = R \cdot lc \tag{3}$$

where ${}_pQ$ = Shear force acting on the panel zone; d_k=lever arm length of panel shear force; P=load at the beam end;R=reaction force at the column end; l_{b2}= distance from loading point to beam face; l_b= beam length ; l_c=column length.

Table 2 shows the panel moment when the panel zone yields ${}_{p}M_{p}$, panel moment when the panel zone reaches the full plastic moment ${}_{p}M_{p}$, panel moment when the beam section reaches yield moment ${}_{p}M_{by}$, panel moment when the attachment reaches the yield moment ${}_{p}M_{ky}$ and panel moment when the column section reaches yield moment ${}_{p}M_{cy}$. Calculation methods of these strength is summerized in Table.3.

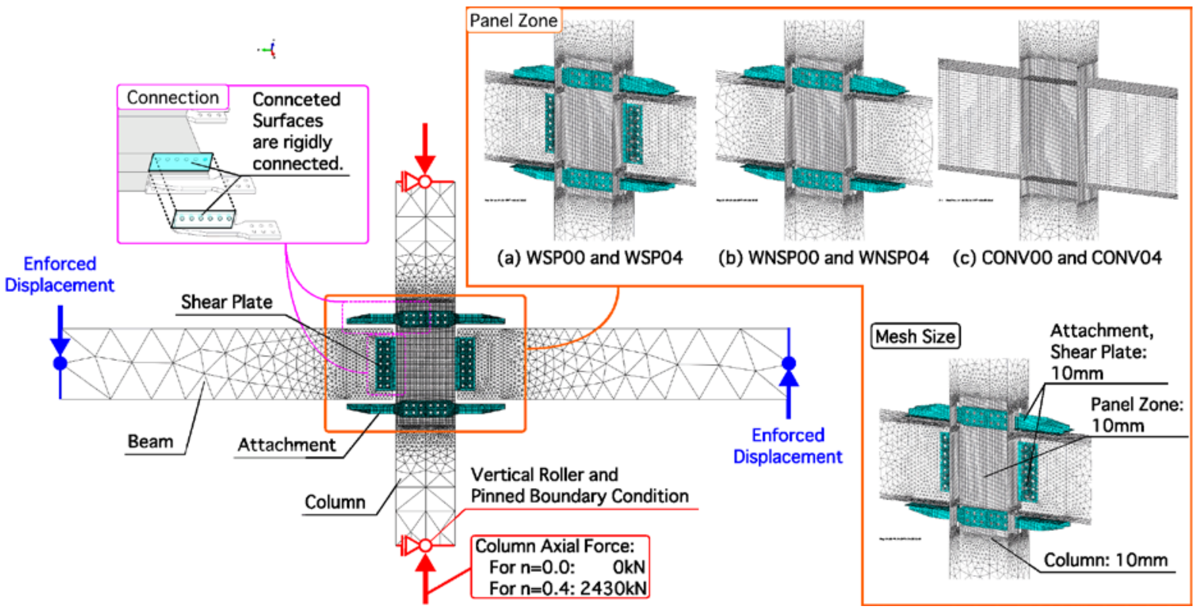


Fig. 5 – FEA model

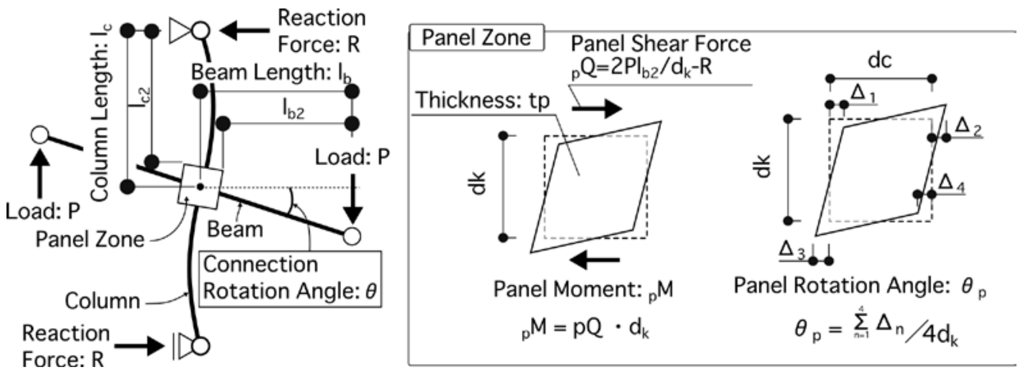


Fig. 6 – Definitions of internal forces and deformations acting on panel zone

Table 2 – Estimated Panel Moments in each model

<table>
<tr><th rowspan="2">Name</th><th colspan="2">Panel</th><th>Beam</th><th>Attachment</th><th colspan="2">Column</th></tr>
<tr><th>Yield
$_pM_y$ (kNm)</th><th>Full Plastic
$_pM_p$ (kNm)</th><th>Yield
$_pM_{by}$ (kNm)</th><th>Yield
$_pM_{ky}$ (kNm)</th><th>Yield
$_pM_{cy}$ (kNm)</th><th>Full Plastic
$_pM_{cp}$ (kNm)</th></tr>
<tr><td>WSP00</td><td rowspan="3">753</td><td rowspan="6">801</td><td rowspan="2">-</td><td rowspan="2">2480</td><td rowspan="3">1990</td><td rowspan="3">2230</td></tr>
<tr><td>WNSP00</td></tr>
<tr><td>CONV00</td><td>2710</td><td>-</td></tr>
<tr><td>WSP04</td><td rowspan="3">690</td><td rowspan="2">-</td><td rowspan="2">2480</td><td rowspan="3">1200</td><td rowspan="3">1310</td></tr>
<tr><td>WNSP04</td></tr>
<tr><td>CONV04</td><td>2710</td><td>-</td></tr>
</table>

Table 3 – Definition of Estimated Strength in each model

Strength	Definition	Symbols
$_pM_y$	$_pM_y = {_pQ_y} \cdot d_k$ $_pQ_y = \frac{d_c \cdot t_p}{\kappa}\sqrt{1-n^2}\,\frac{F_p}{\sqrt{3}}$ $\kappa = \frac{1}{\frac{2}{3}+\frac{4B_c}{d_c}} + \frac{1}{1+\frac{d_c}{6B_c}}$	$_pQ_y$ = panel shear force when the panel zone yields κ =shape factor d_c= column height B_c= column breadth t_p= thickness of the panel zone F_p=yield stress of the panel zone.
$_pM_p$	$_pM_p = {_pQ_p} \cdot d_k$ $_pQ_p = d_c \cdot t_p\left(\frac{F_p}{\sqrt{3}}\right)$	$_pQ_p$=panel shear force when the panel zone reaches the full plastic moment
$_pM_{by}$	$_pM_{by} = {_pQ_{by}} \cdot d_k$ $_pQ_{by} = 2P_{by}\,(l_{b2}/d_k - l_b/l_c)$ $P_{by} = Z_b \cdot F_{by} / l_{b2}$	$_pQ_{by}$ = panel shear force when the beam yields P_{by}=beam end load when the beam yields F_{by}=yield stress of the beam Z_b= section modulus of the beam section.
$_pM_{ky}$	$_pM_{ky} = {_pQ_{ky}} \cdot d_k$ $_pQ_{ky} = 2P_{ky}\,(l_{b2}/d_k - l_b/l_c)$ $P_{ky} = A_k \cdot F_{ky} \cdot d_k / l_{b2}$	$_pQ_{ky}$ = panel shear force when the attachment yields P_{ky}=beam end load when the attachment yields F_{ky}=yield stress of the attachment A_k= cross sectional area in the minimum section of attachment(see Fig. 3(d)).
$_pM_{cy}$	$_pM_{cy} = {_pQ_{cy}} \cdot d_k$ $_pQ_{cy} = 2R_{cy}\,(l_{b2}/d_k\,(l_b/l_c) - 1)$ $R_{cy} = (1\text{-}n)Z_c \cdot F_{cy} / l_{c2}$	$_pQ_{cy}$ = panel shear force when the column yields; R_{by}=reaction force at column end when the column yields; F_{cy}=yield stress of the column; Z_c= section modulus of the column section.

3. Analysis Results

3.1 Comparison of Strength and Stiffness of Panel Zone

Fig. 7 shows the panel moment $_pM$ – panel shear deformation angle γ_p. The definition of γ_p is shown in Fig.6. In each figure, the result of web-clamped type model with shear plate, which corresponds to WSP00 and WSP04, is shown with thick line, the one without shear plate, which corresponds to WNSP00 and WNSP04 is shown with thin gray line and the result of conventional welded type model, which corresponds to CONV00 and CONV04 is shown with dashed line. Moreover, estimated connection strength shown in Table 2 are also shown with horizontal line.

When the results of the web-clamped type and conventional welded type is compared, it is found that the stiffness and strength of web-clamped type are 1.1 times higher than that of conventional welded one. This implies that the panel zone of the web-clamped type has a unique panel shear force transfer mechanism other than the shear panel behavior, which corresponds to that of the conventional welded connection. In order to confirm that, stress distributions over the panel zone are discussed and whether the panel zone of the web-clamped type models shows the shear panel behavior is clarified in the next part.

On the other hand, when the results are compared between the web-clamped type models with and without the shear plate, the models without the shear plate shows earlier strength deterioration than those with the shear plate around γ_p =0.06rad. This implies that the panel zone in the case without thee shear plate failed early due to the difference of the boundary condition around the panel zone. In order to investigate more, deformed shapes of the panel zone when γ_p =0.06rad are investigated in the latter part of the paper.

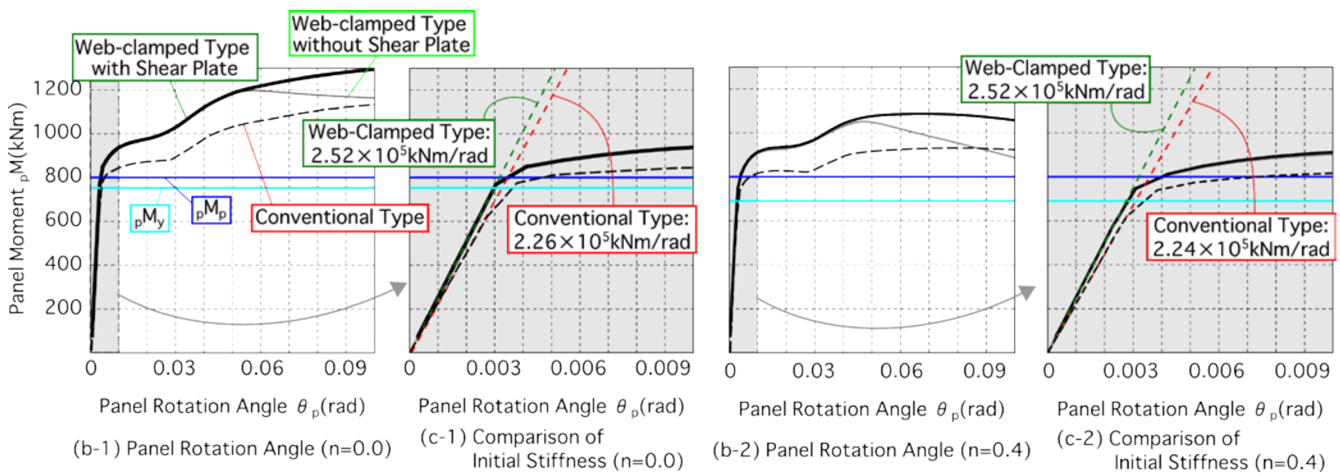


Fig. 7 – Comparison of panel shear deformation angle γ p

3.2 Comparison of Stress and Strain Distribution

In order to investigate whether the panel zone in the web-clamped connection show the shear panel behavior, Fig. 8 shows the shear stress distribution and the plastic strain distribution of the panel zone when the panel moment in each model reaches ${}_pM_p$. Fig. 8(a) and (b) show the shear stress distribution and Fig. 8(c) and (d) show the plastic strain distribution. The range of shear stress shown in Fig. 8(a) and (b) is from 120N/mm^2 to 187N/mm^2, which corresponds to the yield shear stress of the material.

As shown in Fig. 8, shear stress and plastic strain acting on the panel zone is uniform in the conventional welded type models. This results shows that the panel zone behaves as a shear panel. On the other hand, the shear stress and plastic strain acting on the panel of the web-clamped type connection are high at the center of the panel zone. This stress and strain concentration, however is not intense and more than 120N/mm2 of the shear stress and 0.12% of the plastic strain act over the panel zone. Considering the fact that Fig.8 shows the stress and the strain distribution acting on the panel zone in the same panel moment, these observation implies that the panel zone of the web-clamped type connection acts less stress and strain compared to the conventional welded type model. From these observation, one can say that the panel zone of the web-clamped type connection partly shows the shear panel behavior and has another shear force transmission mechanism in addition to the shear panel behavior which is observed in the conventional welded type model.

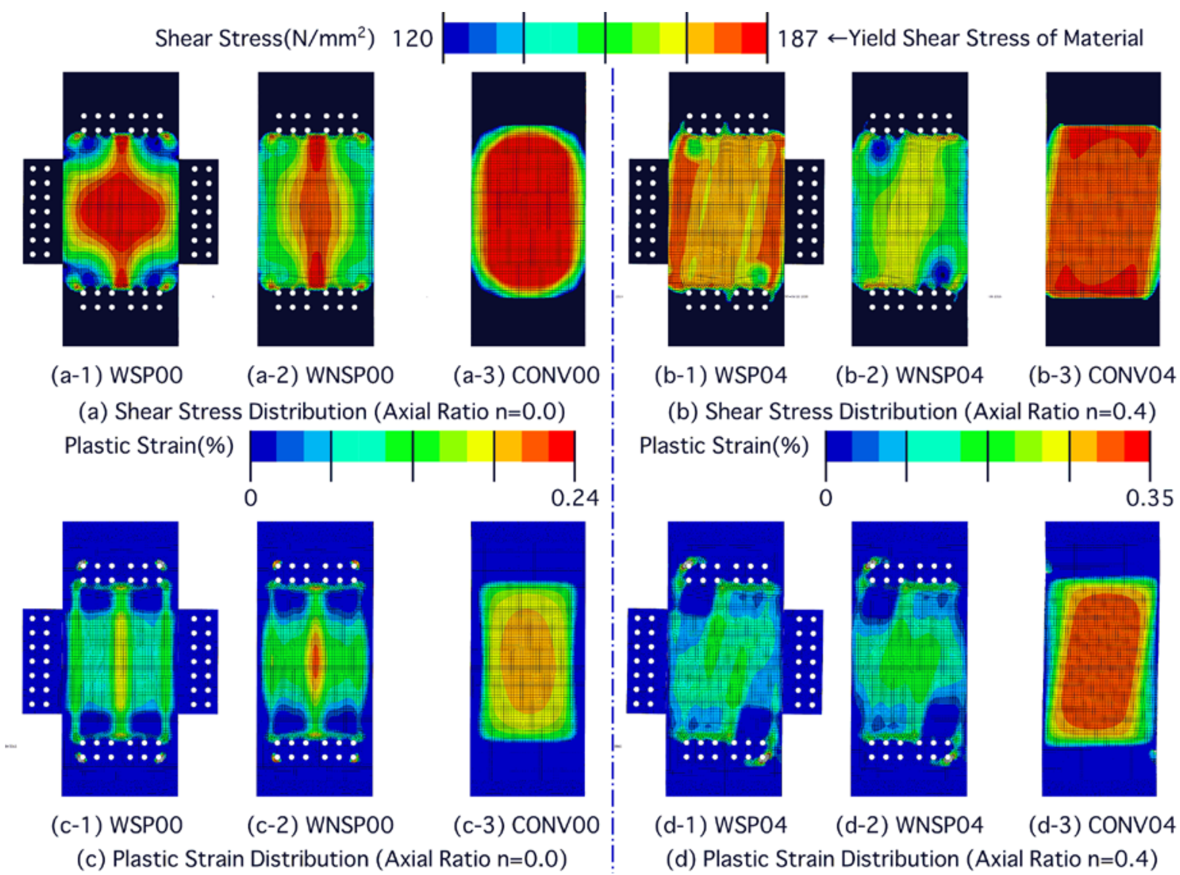


Fig. 8 – Comparison of shear stress distribution and plastic strain distribution at ${}_pM={}_pM_p$

3.3 Comparison of Deformed Shape of Panel Zone

. In order to investigate the panel zone behavior in the large panel shear deformation angle, Fig.9 shows the deformed shape of the panel zone at γ_p =0.06rad. In addition to the deformed shape of the panel zone, the maximum principal stress distributions of the panel zone are presented in the figure in order to investigate whether the truss behavior can be observed over the panel zone of each model.

As shown in Fig.9, diagonal panel shear buckling occurred in all models and tension field is formed diagonally with a square panel zone confined by the vertical and horizontal stiffener in the case of conventional welded connection, or by the attachment and the shear plate in the case of web-clamped type connection. Owing to this tension field, the panel zone forms the truss behavior and resists the panel shear force. As shown in Fig.7, the stiffness of the panel zone is increased after the panel zone yields in all models. This behavior implies the internal force transfer mechanism switched from the shear panel behavior to the truss behavior, which is shown in Fig.9.

Fig.9 also shows that the vertical side of the panel zone deflected only in the web-clamped type models without the shear plate. This behavior might cause the early strength deterioration in the web-clamped type models, which is shown in Fig.7.

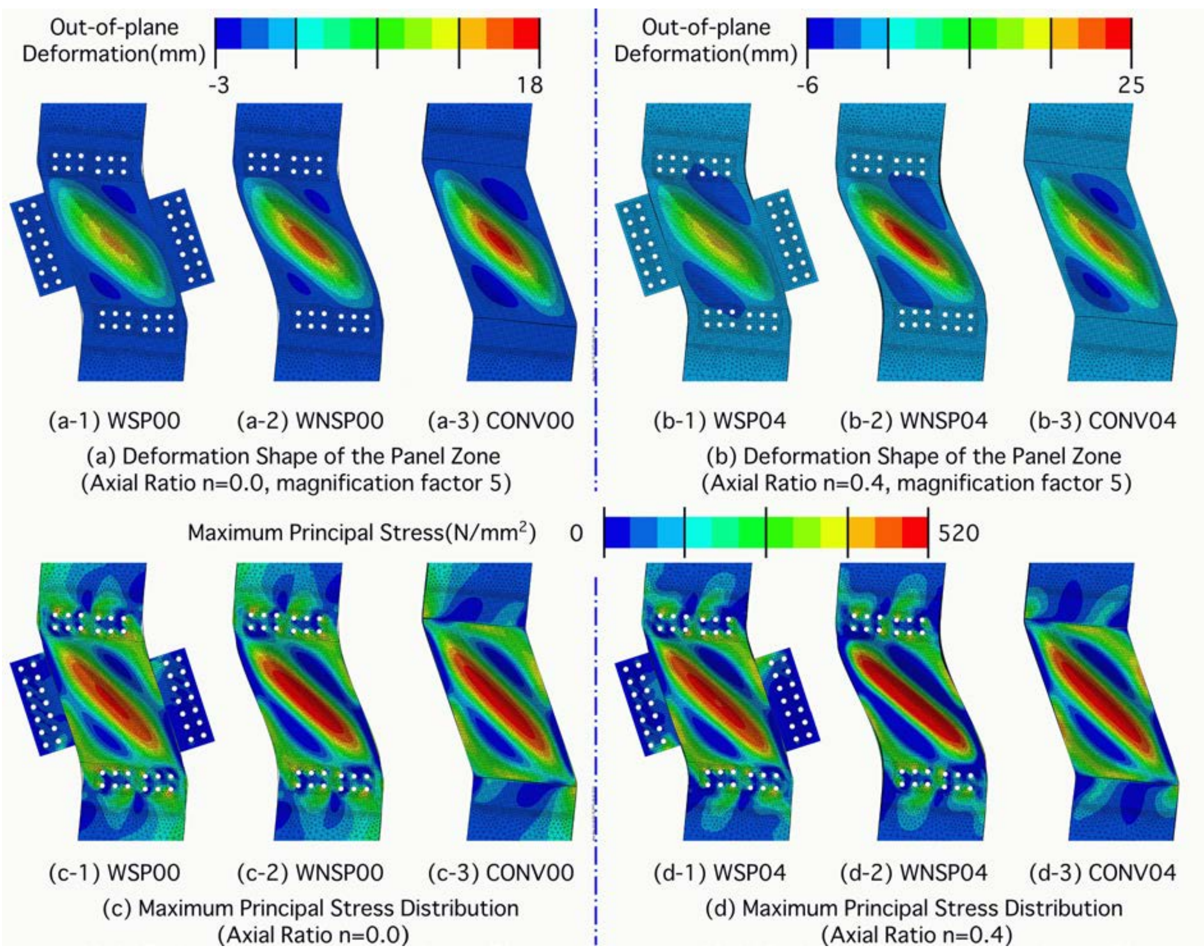


Fig. 9 – Comparison of the deformed shape of the panel zone at γ_p =0.06rad

4. Conclusion

There are two main features of the panel zone in the web-clamped type connection compared to that of the conventional welded connection, which might cause the unique panel zone behavior. Those are: 1) there is no horizontal stiffener in the panel zone, and 2) it is not necessary to connect the beam web and the column by the shear plate. This features may cause the different boundary condition of the panel from that of the conventional welded connection.

In this paper, parametric numerical simulations were conducted and the strain, the stiffness, the stress distribution and the deformation shape of the panel zone were compared between the web-clamped connection and the conventional welded one. In the simulation, the connection type (whether web-clamped type connection or conventional welded type connection), the axial ratio of the column and the existence of the shear plate were varied as parameters, in order to compare the constraint effect of the panel zone.

As a result, when the strength and the stiffness of the panel zone was compared through the shear deformation angle of the panel zone between web-clamped models and conventional welded models, it was observed that the strength and stiffness of the panel zone of the web-clamped type models were 1.1 times higher than those of the conventional welded type models.

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When deformation shape of the panel zone in the large panel shear deformation angle was compared, all the models showed the diagonal panel shear buckling and the diagonal tension field. This may imply that the internal force transfer mechanism of the panel zone of web-clamped connection switches to truss behavior after the panel zone yields, which is the same behavior as that of the conventional welded connection. Especially in the web-clamped type models without the shear plate, deflection in the vertical side of the panel zone was observed, which may cause the early strength deterioration of the panel zone.

5. References

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