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EXPERIMENTAL STUDY ON INELASTIC BEHAVIOR AND ULTIMATE STRENGTH OF STEEL BEAM-TO-COLUMN CONNECTIONS WITH BOLTS AND ANGLES

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SUMMARY

24 steel semi-rigid connection specimens are designed and tested under tension and bending loading conditions in order to investigate elastic and inelastic behavior and establish methods of strength calculation for semi-rigid steel beam-to-column connections with top and seat and double web angles. In most of tests, semi-rigid steel beam-to-column connections demonstrated strong ability of large deformation. Based on test results, tension and bending strength equations are derived for various failure modes. A comparison with measured results in test shows that tension and bending strength equations have accuracy with an error of less than 10%.

INTRODUCTION

Welded connections, namely rigid connections, are widely used in rigid joints of beams and columns. Reinforcement of diaphragms in such welded connections is an effective way of obtaining higher stiffness and strength. But use of diaphragms increases structural complexity, construction time and cost. Welded rigid connections may also cause fracture problems resulting from poor welding quality. In fact, many damaged fully-welded connections were discovered in Hyogo-ken-Nambu earthquake of Japan (January 17, 1995).

Besides rigid connections there is another type of beam-to-column connection, so called semi-rigid connection. In semi-rigid connections, angles and bolts are used to transfer structural forces and moments between beams and columns. There are some advantages and disadvantages in using semi-rigid connections. One of the advantages is that the semi-rigid connection can overcome some manufacture tolerance difficulties and make assembly easier and achieve a better production quality. Another advantage is that connection stiffness and strength could be well designed and controlled by carefully selecting a combination of angles and bolts. Therefore, the desired failure mode can be obtained and scheme of energy absorption can be well manipulated. A disadvantage of using semi-rigid connections is that it may reduce the stiffness of the connection and increase deformation of the connection. In Japan,

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semi-rigid connections are mostly used in prefabricated low-rise residential buildings, usually braced frames.

In this study, 24 semi-rigid connection specimens were designed, fabricated and tested for their elastic and plastic behavior. In tests, the thickness of angles, diameter of bolts, location of bolts are chosen as design parameters, failure modes, strength, stiffness, hysteresis characteristic curves are observed and recorded as test results. Some practical formulas for strength calculations were developed based on test results and observations.

TESION TESTS OF ANGLES

Tension test specimens and test procedure

This paper focuses on semi-rigid beam-to-column connections with top, seat, and double web angles. Top and seat angles are most important in increasing structural stiffness and strength. Since tension and compression are dominant in top and seat angles, tension tests are conducted prior to bending tests. A tension test specimen is illustrated in Figure 1. A specimen is made of 4 angles of same dimension, two tension plates of same dimension and a one compression plate. Tension plates could be viewed as flanges of beam in semi-rigid beam-to-column connections. Compression plate restraints the deformation of angles. An Amsler type universal testing machine is employed to apply a tensile load to specimen. All specimens are loaded until failure, in which angle or bolt will fracture.

All angles are made of JIS SS400 steel. 4 different thicknesses used are 9 mm, 12 mm, 15 mm and 20 mm. Two type of high strength bolts used are M20 and M22 of class F10T. Information of all 15 tension test specimens is given in Table 1.

Tension test results

Figure 2 shows load versus displacement curves from tension tests. Important mechanical properties of specimen such as initial stiffness ${}_eK_i$, and yielding strength ${}_eT_y$ and ultimate strength ${}_eT_u$ are observed or derived from those load-displacement curves and listed in Table 2. ${}_eT_u$' in Table 2 is the effective ultimate strength which is measured when the specimen is stretched by 30 mm. Although tensile strength is still increase after specimens stretches more than 30 mm, additional tensile strength increase will not be useful because actual structures won't allow that much deformation.

Some specimens failed in angles and some failed in bolts. But all specimens stretched by more than 30 mm and exhibit their strong ability of deformation. Except specimens A20-12, D22-20, E22-15, all specimens illustrated similar ability of large deformation. Specimens of A20-12, D22-20 and E22-15 failed because of quick fracture of bolt. Those bolt-failed specimens showed relative higher yielding strength. More than 50% of the specimens showed their ultimate strength to be 3 times higher than their yielding strength.

Table 1. Details of tension test specimens

Specimen	Angle	Bolt	Width (mm)	g1 (mm)	g3 (mm)
A20-9	L-150×100×9	M20	150	50	90
A20-12	L-150×100×12	M20	150	50	90
B20-9	L-150×100×9	M20	150	60	90
B20-12	L-150×100×12	M20	150	60	90
B22-12	L-150×100×12	M22	150	60	90
C20-12	L-175×120×12	M20	150	70	80
C20-15	L-175×120×15	M20	150	70	80
C22-12-1	L-175×120×12	M22	150	70	80
C22-12-2	L-175×120×12	M22	175	70	95
C22-12-3	L-175×120×12	M22	175	70	115
C22-15	L-175×120×15	M22	150	70	80
D22-12	L-175×130×12	M22	175	80	95
D22-20	L-190×130×20	M22	150	80	80
E22-15	L-175×120×15	M22	175	65	95
F22-20	L-190×130×20	M22	150	90	80

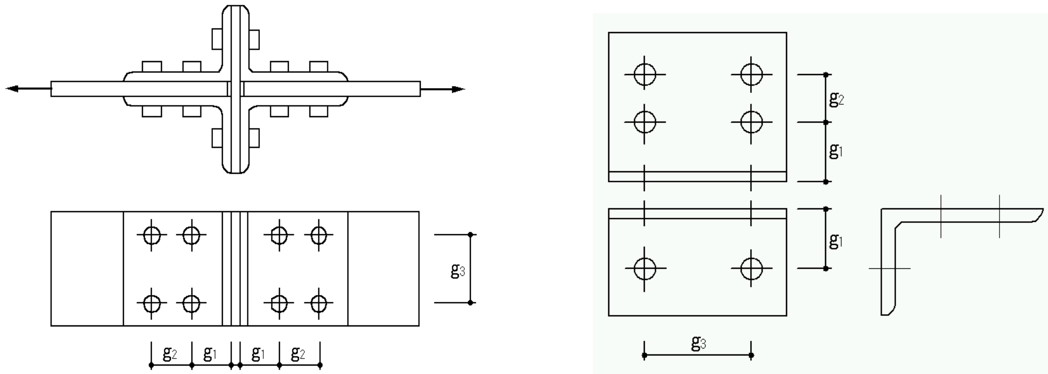


Fig. 1. Tension test specimen

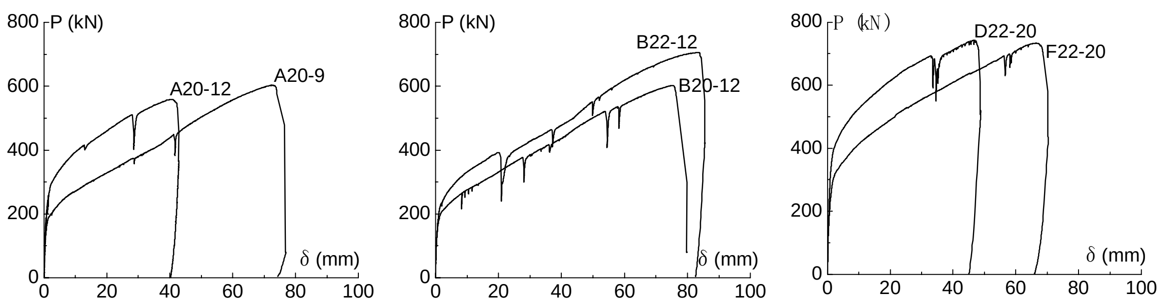


Fig. 2. Load-displacement curves from tension tests

Table 2. Results of tension tests

Specimen	Test						Calculated strength		Number
	${}_eK_i$ (kN/mm)	${}_eT_y$ (kN)	${}_eT_u'$ (kN)	${}_eT_u$ (kN)	${}_e\delta_{max}$ (mm)	Failure	T_y (kN)	T_u (kN)	
A20-9	143	185	380	604	73	Bolt	213	419	1
A20-12	156	279	514	559	41	Bolt	324	546	2
B20-9	82	124	259	596	88	Angle	139	250	3
B20-12	144	195	385	602	75	Bolt	240	373	4
B22-12	147	227	428	706	83	Bolt	260	519	5
C20-12	90	178	336	600	79	Bolt	177	339	6
C20-15	162	272	475	663	66	Bolt	288	471	7
C22-12-1	165	187	363	752	93	Bolt	188	363	8
C22-12-2	125	227	414	772	92	Bolt	220	423	9
C22-12-3	120	218	417	748	90	Angle	220	423	10
C22-15	163	295	532	724	67	Bolt	307	503	11
D22-12	115	180	335	738	94	Bolt	168	312	12
D22-20	229	380	678	743	47	Bolt	410	736	13
E22-15	293	410	691	751	46	Bolt	390	680	14
F22-20	164	301	554	733	67	Bolt	356	630	15

Two types of failure were observed in tension tests. One is bolt fracture and another is angle fracture. Bolts had two types of fracture. One is that bolt fracture occurred after large deformation of angles. Another is that bolt fracture occurred with only a small deformation. Based on those observations, 3 failure modes could be stated as (1) angle fails because of excessive deformation (2) bolt fails prematurely (3) both angles and bolts experience large deformation and one of them eventually fails.

It is observed that the thickness of angles, the diameter of bolts and the center distance of bolts is primary factor in determination of the yielding strength, the ultimate strength and the initial stiffness of a specimen. Increase in thickness of angle usually increases the ultimate strength of angle. But sometimes it may reduce ultimate strength if it induces prematurely bolt failure as illustrated by specimen A20-12. Therefore accurate prediction of failure mode is a condition for an accurate strength prediction of a semi-rigid connection.

BENDING TEST OF SEMI-RIGID BEAM-TO-COLUMN CONNECTION

Bending test specimens and test procedure

The bending test setup is illustrated in Figure 3. An H-shape beam is connected to the base block at the bottom through angles and bolts. A horizontal actuator is connected to the top of the beam to apply a horizontal force. In this study, column deformation is ignored and the base block is made very rigid compared with the vertical beam.

Figure 4 and Table 3 describe detailed design variables in bending test specimen. In order to investigate contribution from different angles, some test specimens only used top and seat angles and some test specimens only had double web angles. Five types of bending specimens are (1) FWa and FW type of semi-rigid connections having top angle, seat angles, and double web angles (2) F type of semi-rigid connections having top angle and seat angles (3) W and Ws type of semi-rigid connections having only double web angles. Each of the angles in Ws type was split in three pieces to make three independent small angles. Beam used in test is of JIS steel grade SS400 rolled H-shape section, H-300x150 x 6.5 x 9 and H-350x175x7x11. All angles are made of JIS steel grade SS400 and angle of L-200x130x15 was made by cutting L-200x200x15. FWa50 and Fw50 specimens used high strength bolts of M16 of class F10T, which has a pretension of about 119 KN. Other types of specimens used M22 of class S10T, which has a pretension of about 221 KN. Bolt holes have clearance of 2 mm.

Table 3. Details of bending test specimens

<table>
<tr><th rowspan="2">Specimen</th><th rowspan="2">Beam</th><th rowspan="2">Angle</th><th rowspan="2">g_1 (mm)</th><th rowspan="2">Bolt</th><th colspan="3">Top and seat angle</th><th colspan="3">Web angle</th></tr>
<tr><th>n_t</th><th>Width (mm)</th><th>g_3 (mm)</th><th>n_t</th><th>Width (mm)</th><th>g_5 (mm)</th></tr>
<tr><td>FWa50-M
FWa50-C</td><td rowspan="2">H-300×150
×6.5×9</td><td>L-90×90×7</td><td rowspan="2">50</td><td rowspan="2">M16</td><td rowspan="2">2</td><td rowspan="2">150</td><td rowspan="2">90</td><td rowspan="2">3</td><td rowspan="2">220</td><td rowspan="2">70</td></tr>
<tr><td>FW50-M
FW50-C</td><td>L-125×90×10</td></tr>
<tr><td>FW70-C</td><td rowspan="4">H-350×175
×7×11</td><td rowspan="4">L-200×130×15</td><td rowspan="3">70</td><td rowspan="4">M22</td><td rowspan="2">2</td><td rowspan="2">175</td><td rowspan="2">95</td><td>3</td><td>270</td><td>90</td></tr>
<tr><td>F70-C</td><td>---</td><td>---</td><td>---</td></tr>
<tr><td>W70-C
Ws70-C</td><td>---</td><td>---</td><td>---</td><td>3</td><td>270</td><td>90</td></tr>
<tr><td>FW80-C</td><td>80</td><td>2</td><td>175</td><td>95</td><td>3</td><td>270</td><td>90</td></tr>
</table>

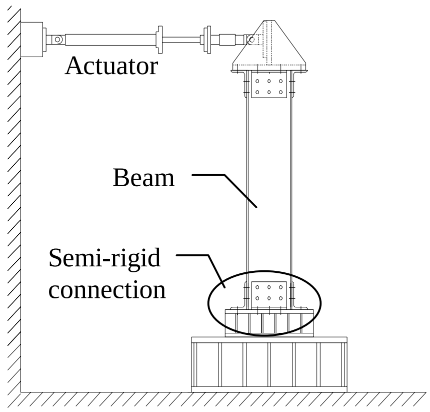
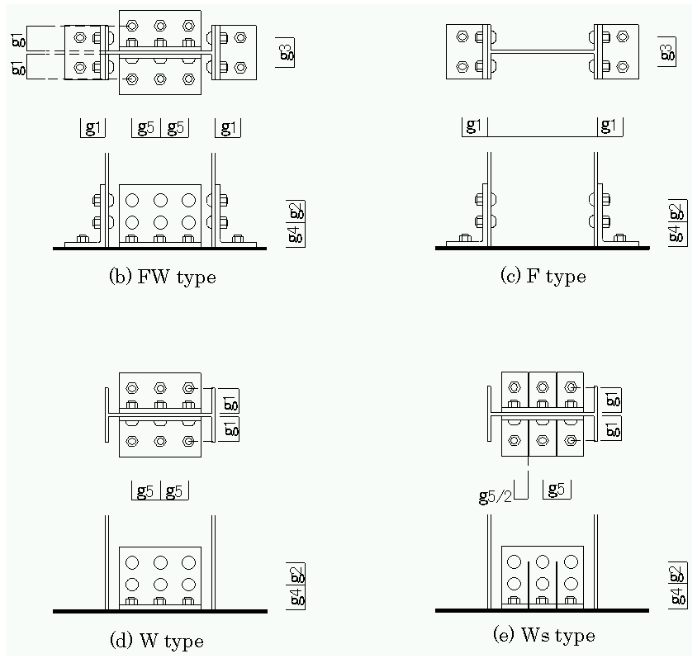


Fig. 3. Test setup



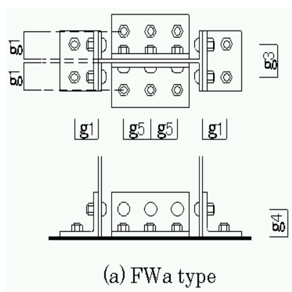


Fig. 4. Details of semi-rigid connections

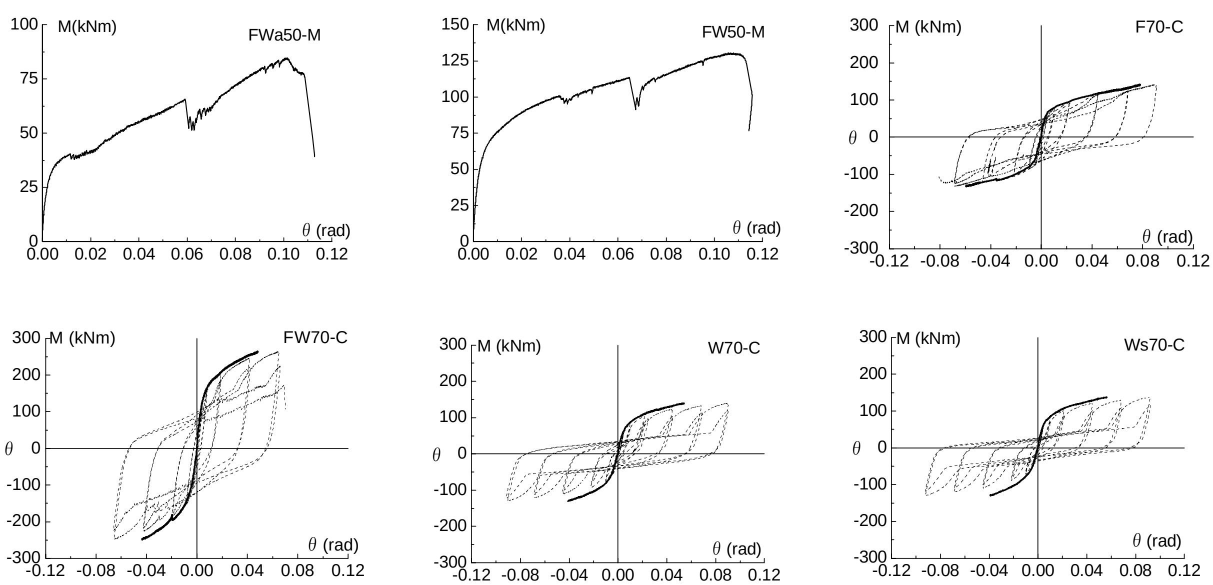


Fig. 5. Moment-rotation curves from bending tests

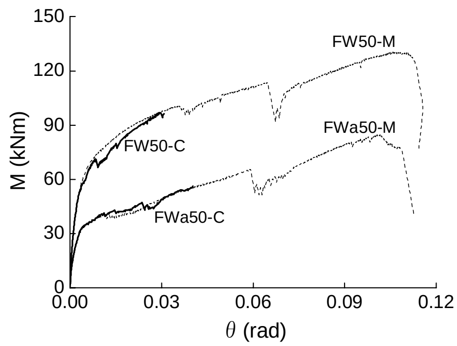


Fig. 6. Comparison between monotonic and cyclic loading tests

Bending test results

Beam used in bending tests was designed so strong that no plastic deformation was developed in the beams under testing loading conditions. Horizontal displacement at the loading point was measured. Rotation of semi-rigid connection is calculated based on total displacement of loading point by subtracting calculated displacement of elastic beam. First two plots in Figure 5 shows two monotonic moment-rotation curves. Frictional slip in bolt connection was observed only after semi-rigid connection specimen reached the yielding point. This small frictional slip didn't have much influence in moment-rotation curve until rotation became more than 1/20 rad. Number of frictional bolts in FW50 was twice of that in FWa50. Test results show the load, at which FW50 had begun slipping, was twice as that in FWa50. Moment-rotation curves held a nearly linear relationship after yielding. This observation suggested that a bilinear moment-rotation relation could be assumed. Specimens using 7 mm angles fractured in fillet while specimens using 10 mm angles fractured in bolts.

Cyclic loading moment-rotation curves are shown in Figure 5. Loading was controlled by applying a displacement instead of applying a force. Same displacement was repeated twice before moving to next loading level. A test started with an initial displacement then increased by ±2, ±4, ±8, ±12, …, times until specimen failed. A skeleton curve was also plotted as bold in each cyclic curve plots in Figure 5. Figure 6 shows comparison between monotonic loading and cyclic loading skeleton curve. Difference between the skeleton curve and the monotonic curve for same type of specimen is very close. This means that the strength calculated used the skeleton curve and strength calculated using the monotonic curve are almost same and the difference is negligible. Table 4 listed test results of the initial stiffness ${}_eK_i$, the bending yielding strength ${}_eM_y$, the ultimate bending strength ${}_eM_u$ and the ultimate rotation ${}_e\theta_{max}$. ${}_eM_u$' in Table 4 is the effective ultimate strength, which was measured when the semi-rigid connection rotated by 1/20 rad.

Pinching effect of hysteresis characteristic were observed from cyclic loading moment-rotation curves, and it makes building restoring force model more difficult. In most of specimens using M22 bolts, frictional slips were not observed due to higher pretension force in M22 bolts, except FWa50-C and FW50-C. As mentioned earlier, in all specimen tests, beam worked in elastic range. Inelastic deformation occurred in connection area. Angles and bolts absorbed all energy. It is as expected that all specimens exceeded 1/20 rad rotation and exhibited their ability for a large deformation. It was noticed that the hysteresis curve takes a reversed S-shape when looked in loading direction. W type specimens with only double web angles showed more pinching effect than F type specimens with only top and seat angles. W type specimens with or without slit didn't show much difference in the initial stiffness and the strength and showed a very small difference in energy dissipation. As a result, web angles can be treated as pure tension or comparison components similar to top and seat angles. This treatment can simplify calculation and make strength formula more practical.

Table 4. Results of bending tests

Specimen	Test						Calculated strength		Number
	${}_eK_i$ (kNm/rad)	${}_eM_y$ (kNm)	${}_eM_u'$ (kNm)	${}_eM_u$ (kNm)	${}_e\theta_{max}$ (rad)	Failure	M_y (kNm)	M_u (kNm)	
F70-C	14077	67	123	137	0.093	Angle	81	128	1
W70-C	12180	72	133	135	0.091	---	71	143	2
Ws70-C	12725	68	132	133	0.092	---	71	143	3
FWa50-M	9915	30	61	85	0.101	Angle	30	59	4
FWa50-C	9245	33	56	56	0.058	Angle			
FW50-M	13834	62	107	131	0.107	Bolt	63	105	5
FW50-C	16323	55	97	97	0.056	Angle			
FW70-C	25587	146	256	256	0.077	Angle	152	271	6
FW80-C	21259	119	220	225	0.067	Angle	126	207	7

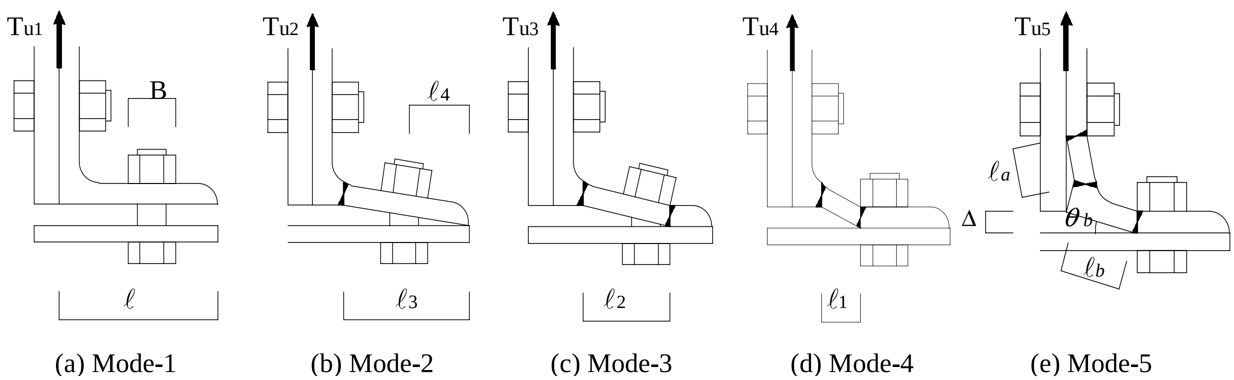


Fig. 7. Failure modes for semi-rigid connection

PROPOSED METHODS FOR STRENGTH CALCULATION

Ultimate tensile strength of angles

In order to establish ultimate bending strength for semi-rigid connections, ultimate tensile strength of top and seat angles should be discussed first. Based on test results, four failure modes with small deformation and one failure mode with large deformation are assumed. Those five failure modes are illustrated in Figure 7. For thinner angle, tensile deformation in bolts is small and won't create gap between angle and column. Thinner angle will fail when two plastic hinge forms on the inner side of bolt as illustrated in failure mode 4. As angle thickness increases, the gap between angle and column will become bigger and failure modes will change to mode 3 in which one plastic hinge will form on each side of a bolt. With further increase of angle thickness, one plastic hinge and bigger gap will cause a failure as illustrated in failure mode 2. When angle is very strong, bolt stretches long such that a complete gap between angle and column will be created as illustrated in failure mode 1. The ultimate strength corresponding to each failure mode can be calculated using equation (1). In general, the minimum strength among all calculations for all failure modes should be figured out as the ultimate strength of the semi-rigid connection.

$$\left.\begin{array}{ll} T_{u1} = n_t \cdot N_u \;, & T_{u2} = \dfrac{(M_{au} + T_{u1} \cdot \ell_4)}{\ell_3} \\ T_{u3} = \dfrac{(4M_{au} + T_{u1} \cdot B)}{2\ell_2} \;, & T_{u4} = \dfrac{2M_{au}}{\ell_1} \end{array}\right\} , \quad T_u = \min(T_{u1},\, T_{u2},\, T_{u3},\, T_{u4}) \tag{1}$$

$$if \;\; (T_u = T_{u4} \;\; and \;\; T_{u4} < 0.8T_{u3}) \;\; \rightarrow \;\; T_u = T_{u5} = \frac{2M_{au}}{\ell_b}\left(\frac{1}{\cos\theta_b} + \frac{t_a + r}{2\ell_a} + \frac{\ell_b}{\ell_a}\frac{(1-\cos\theta_b)}{\sin\theta_b}\right) \tag{2}$$

When angle is experiencing large deformation, the calculated strength using equation (1) could be smaller than actual strength. In order to improve the accuracy of calculated results, equation (2) is proposed to include large deformation. If the conditions are met in equation (2), equation (2) should be used to calculate the ultimate strength which is associated with failure mode 5.

In above equations, n_t is number of tension bolts, N_u is the ultimate tensile strength of one bolt, $\ell_1 = g_1 - t_a - (r+B)/2$, $\ell_2 = g_1 - t_a - (r-B)/2$, $\ell_3 = \ell - t_a - r/2$, $\ell_4 = \ell - g_1$, $\ell_a = g_4 - t_a - (r+B)/2$, $\ell_b = g_1 - (t_a + B)/2$, ℓ is the length of angle, t_a is the thickness of angle, r is the radius of fillet of angle, M_{au} is the bending strength of angle. B is the diameter of bolt head or nut. θ_B is an angle calculated from the 1/20 rad rotation of semi-rigid connection.

Ultimate bending strength of semi-rigid connection

Ultimate bending strength of a semi-rigid connection with top and seat angle

Ultimate bending strength equation (3) and (4) are derived from moment equilibrium conditions about rotation center as illustrated in Figure 8. In equation (4), H_b is beam depth.

$$\left.\begin{array}{ll} M_{jfu1} = T_{u1}\left(H_b + g_1 + 0.5t_a\right) + M_{au} \,, & M_{jfu2} = T_{u2}\left(H_b + 1.5t_a + 0.5r\right) + 2M_{au} \\ M_{jfu3} = T_{u3}\left(H_b + 1.5t_a + 0.5r\right) + 2M_{au} \;, & M_{jfu4} = T_{u4}\left(H_b + g_1 + 0.5t_a - 0.5B\right) \end{array}\right\} \tag{3}$$

$$\left.\begin{array}{l} M_{jfu} = \min(M_{jfu1},\, M_{jfu2},\, M_{jfu3},\, M_{jfu4}) \\ if \;\; (M_{jfu} = M_{jfu4} \;\; and \;\; M_{jfu4} < 0.8M_{jfu3}) \;\; \rightarrow \;\; M_{jfu} = M_{jfu5} = T_{u5}\left(H_b + g_1 + 0.5t_a - 0.5B\right) \end{array}\right\} \tag{4}$$

Ultimate bending strength of a semi-rigid connection with double web angles

Based on bending test results, web angles will be simplified as pure tension angles. In order to calculate bending strength, stress distribution is assumed as shown in Figure 9 (a). Stress distribution in web takes the same shape as of load-displacement curve in angle tension test, which is represented by the dotted

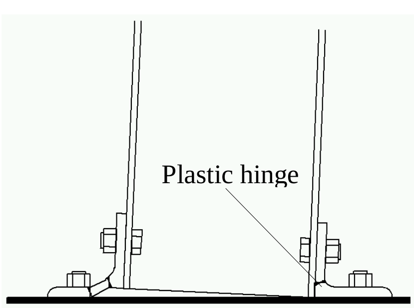
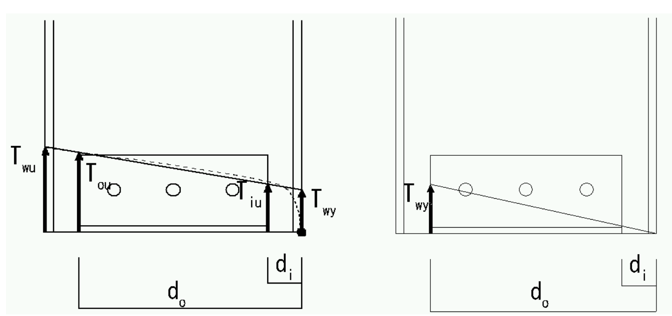


Fig. 8. Failure mode



(a) Ultimate strength (b) Yield strength

Fig. 9. Stress distribution

line in Figure 9 (a). In ultimate strength calculation, the ultimate tensile strength of web angle T_{wu} is assumed on the left side and the yielding tensile strength T_{wy} is assumed on the right side. A straight line is used to connect T_{wu} to T_{wy}. The ultimate bending strength is derived as

$$M_{jwu} = \{T_{ou} \cdot (2d_o + d_i) + T_{iu} \cdot (d_o + 2d_i)\}/3 \tag{5}$$

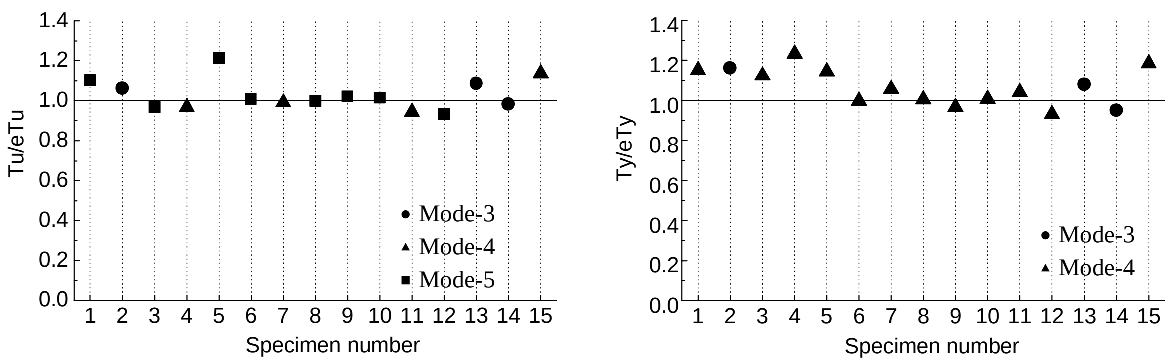
Ultimate bending strength of a semi-rigid connection with top, seat and double web angles

The ultimate bending strength of this type of semi-rigid connection is sum of the ultimate strength of top and seat angles and double web angles, which is given in Equation (6).

$$M_{ju} = M_{jfu} + M_{jwu} \tag{6}$$

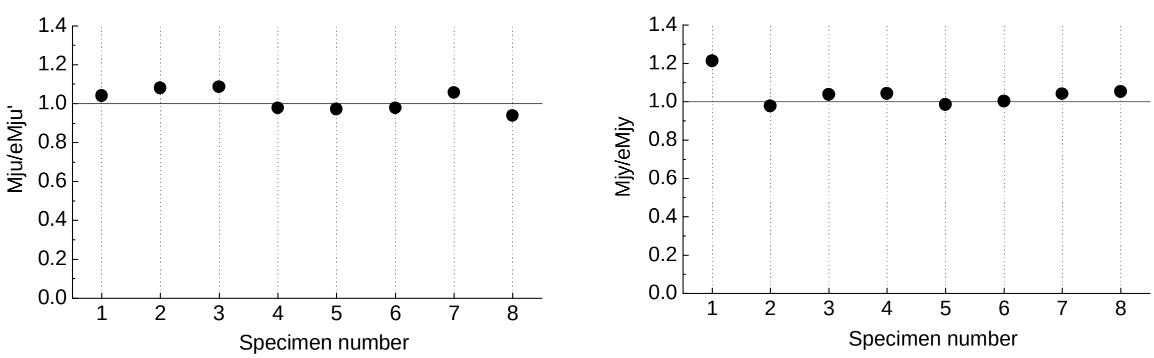
Yield bending strength of semi-rigid connection

In yielding strength calculation of bending, large deformation shouldn't be considered. But all strength equations can still be used if replacing the ultimate bending moment M_{au} by the plastic bending moment M_{ap} and replacing the bolt ultimate strength N_u by the bolt opening strength N_s and replacing T_{ou} and T_{iu} by the yielding strength of angle T_{wy} and $T_{iy} = T_{wy} \cdot d_i / d_o$.



(a) Ultimate strength (b) Yield strength

Fig. 10. Comparison of calculated tensile strength with test results



(a) Ultimate strength (b) Yield strength

Fig. 11. Comparison of calculated bending strength with test results

Accuracy in semi-rigid connection strength calculation

Calculated strength of tension test specimens using equations proposed in this paper is compared with measured strength of tension from tests. Strength comparison can be found in Table 2 and Figure 10. Except specimen B22-12 and F22-20, difference between the calculated ultimate strength of tension and the measured ultimate strength of tension are within 10%. Difference between the calculated yield strength of tension and measured yield strength of tension is bigger than the difference between the ultimate. One third of specimen shows that calculated yield strength of tension is more than 10% higher than measured yield strength of tension.

Calculated strength of bending test specimens using equations in this paper is compared with measured strength of bending in tests. Strength comparison can be found in Table 4 and Figure 11. For FWa50 and FW50, average test results from monotonic test and cyclic test is used. Differences between the calculated ultimate strength of bending and measured ultimate strength of bending are within 10%. Differences between calculated yielding strength of bending comparison show similar accuracy except specimen F70-C. It is very interesting to notice that semi-rigid connections with top and seat angles and double web angles even have better accuracy.

CONCLUDING REMARKS

(1). All specimens exhibited a strong ability of a large deformation even though failure modes could be different.

(2). Failure mode in Semi-rigid connection could be classified into three types. (1) angles fail due to excessive deformation, (2) bolts fail prematurely and (3) both angle and bolt experiences large deformation and angle or bolt eventually fails due to excessive deformation. Failure mode (1) and (2) exhibits remarkable strength hardening behavior after yielding.

(3). When large deformation is included, accuracy in strength calculation is improved in semi-rigid connection with top and seat angles and double web angles.

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