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Effect of collapse modes on ductility of steel frames

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ABSTRACT: This paper presents the results of an experimental work carried out to study the behavior of steel frames under earthquake loading. Test parameters are the collapse modes of steel frames and presence of scallops at beam end. From the test results, the effect of these parameters on the behavior of steel frames are discussed. It is shown that the frame specimen with non-scallop and of mode where both beam and beam-to-column connection panel collapse simultaneously in flexure and in shear respectively, possesses the best earthquake resistant properties in all frame specimens.

1 INTRODUCTION

In conjunction with the plastic and seismic design, it has been made clear that the behavior of steel frames under earthquake loading is affected by bucklings of members, mechanical properties of steel and fracturing at connections.

In general, beam-to-column connection panels in steel frames are designed not to yield prior to yielding of the surrounding members under earthquake loading. Nevertheless, the frames based on this design method don't always show large ductility. Strength, ductility and energy dissipation capacity of steel frames are affected by collapse modes of frames. However, few researches have been reported on the effect of different collapse modes on the behavior. Then in this paper, the behavior of steel frames with three collapse modes is discussed, where the first mode is that beam-to-column connection panel collapses in shear, the second, beam member collapses in flexure, and the third, both beam and connection panel collapse simultaneously in flexure and in shear, respectively.

Recently, it has been pointed out that scallops lead to a problem for the ductility of steel building frames. In case that a beam member is welded to a column member at the beam-to-column connection, the scallops are usually located at the beam end, in order to avoid the intersection of the fillet-welding line of the beam web and the back-running line of the beam flange. If the scallops are located at the beam end, the beam flange is apt to fracture and then the strength of the frame deteriorates rapidly, under large earthquake excitation. In view of this phenomenon, specimens with non-scallop were tested in addition to specimens with scallops, and an effect of the presence of the scallops on the behavior of steel frames is discussed, too.

2 TEST PROGRAM

The test model is a steel T-shaped frame which is designed by considering the stress distribution of a building frame subjected to vertical and horizontal loads (see Fig.1).

The experimental parameters are the collapse modes and the presence of the scallops at the beam end.

Collapse modes are as follows ;

Mode P: Frame where beam-to-column connection panel collapses in shear

Mode B: Frame where beam member collapses in flexure

Mode S: Frame where both beam member and connection panel collapse simultaneously in flexure and in shear, respectively

The shape and dimensions of the specimen are shown in Fig.2. Built-up wide flanges were used for beam and column members of the specimens. As for the beam-to-

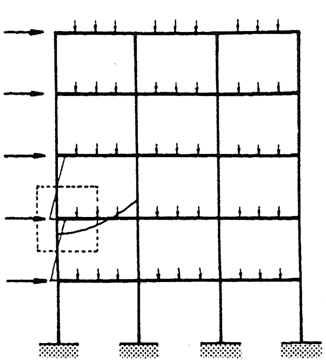


Fig.1 Test Model

Table 1 Mechanical Properties of Steel Plates

Member	element	σy (kN/mm^2)	σu (kN/mm^2)	εy (%)	εst (%)	εb (%)
Beam	Flange	295	421	0.152	2.80	31.8
	Web	291	409	0.167	2.31	27.8
Column	Flange	272	431	0.137	2.17	29.7
	Web	294*	431	0.333	-	28.6

σy: Yield point stress σu: Tensile strength
εy: Strain at yield point εst: Strain at start of strain hardening
εb: Maximum elongation
* σy of column web is obtained through the 0.2% offset method.

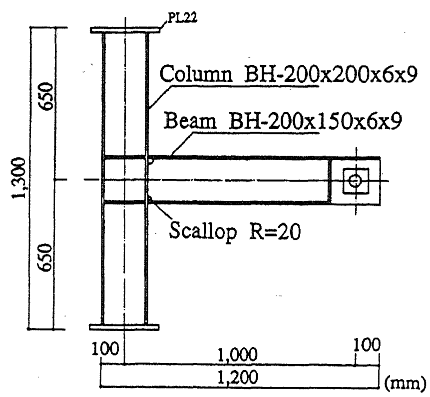


Fig.2 Specimen

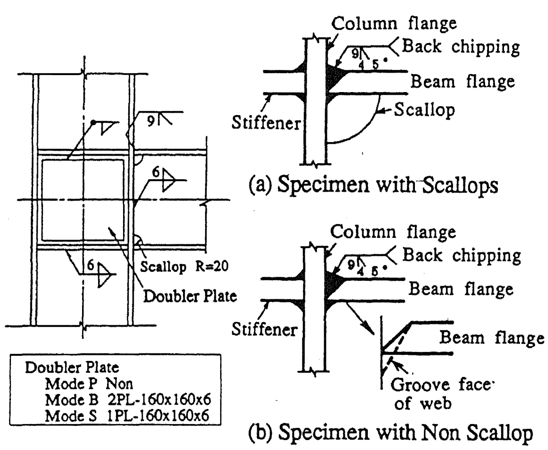


Fig.3 Beam-to-Column Connection Panel

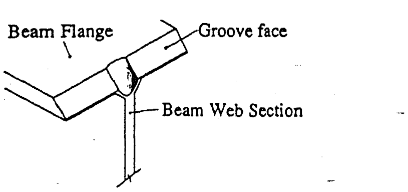


Fig.4 Groove Face of Beam with Non Scallop

column connection, the beam member was connected to the column member by welding (see Fig.3). The mechanical properties of steel plates used for specimens are indicated in Table 1.

The collapse modes are controled by changing the ratio of the flexural strength of the beam member $_{B}Pp$ to the shear strength of the beam-to-column connection panel $_{p}Pp$, where $_{B}Pp$ is the beam load when the moment of the beam at the column face equals to the full plastic moment of the beam section and $_{p}Pp$ is the beam load converted from the shear strength of the connection panel $_{p}Mp$.

$$_{p}Mp = Ve\frac{\sigma y}{\sqrt{3}} \qquad (1)$$

where, Ve=Volume of the beam-to-column connection panel(=$_{B}h \cdot {}_{C}h \cdot t_w$), σy=Yield stress of the steel, $_{B}h$ and $_{C}h$=Height of the beam section and the column section, respectively and t_w=Plate thickness of the connection panel.

The beam-to-column connection panel is reinforced by fillet-welding one or two doubler plates 6mm thick, for the Mode S or the Mode B, respectively.

Figure 3 shows the details of the beam-to-column connection. The beam flange was connected to the column flange by butt-welding and back-running, and the beam web was done by fillet welding. For the specimen with scallops, there are scallops (radius 20mm) at the beam end, in order to avoid the intersection of the back-running line and the fillet-welding line. On the other hand for the specimens with non-scallop, the beam flange cannot be back-run at the junction of the flange and web at the beam end, so a part of the web at the beam end was cut as a groove and was butt-welded to the column flange. The sketch of the groove face of the beam member for specimens with non-scallop is shown in Fig.4.

Total six specimens were tested. The test program is indicated in Table 2. In this table, the beam flexural strength with scallops is calculated by considering the lack of cross section by the scallops, and the column flexural strength $_{C}Pp$ is the beam load when the moment of the column at the beam face equals to the reduced full plastic moment by axial force.

The experimental apparatus is shown in Fig.5. The top and bottom of the column are free to rotation and fixed to sway displacement, so P-δ effect is not introduced in this loading system. The beam member is supported by lateral supporting frame in order to avoid occurrence of the lateral buckling.

The cyclic load P was applied to a beam, under constant vertical load $N=0.3N_0$ on the column top, where N_0 is the compressive strength of the column section. The similar stress distribution of a building frame under earthquake excitation is produced by these loads.

Vertical displacement of the beam loading point and shearing deformation of the connection panel were measured in cyclic loading process.

Table 2 Test Program

Name of Specimen	Collapse Mode	Theoretical Strength			$\frac{_P P_p}{_B P_p}$	Presence of scallops
		Column $_cP_p$ (kN)	Beam $_BP_p$ (kN)	Panel $_PP_p$ (kN)		
MP-N	Connection Panel	195.4	99.3	48.1	0.48	Non
MP-S		195.3	91.2	48.2	0.53	Presence
MB-N	Beam	195.9	98.3	143.9	1.46	Non
MB-S		195.0	91.7	144.3	1.57	Presence
MS-N	Beam and Panel	195.0	98.9	96.2	0.97	Non
MS-S		193.9	90.8	95.2	1.05	Presence

MS-N
— Presence of Scallop
N: Non Scallop , S: Scallop
—Collapse Mode
MP: Mode P , MB: Mode B , MS: Mode S

Table 3 Test Results

Name of Specimen	Maximum strength Pmax (kN)	Theoretical strength Pcal (kN)	$\frac{P_{max}}{P_{cal}}$	Rotation angle R (%)				Ductility factor
				Maximum strength	Local buckling Flange	Local buckling panel	Crack	μ
MP-N	83.6	48.1	1.74	7.0	-	6.5	-	71
MP-S	83.9	48.2	1.74	7.0	-	-	5.5	79
MB-N	136.5	98.3	1.39	3.5	2.5	-	-	34
MB-S	138.8	91.7	1.51	3.5	2.5	-	4.0	30
MS-N	133.0	98.9	1.34	6.5	-5.0	-	-	90
MS-S	122.5	90.8	1.35	4.0	-3.0	-	4.0	60

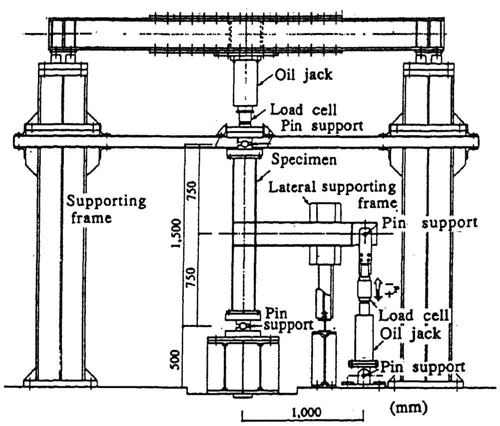


Fig.5 Experimental Apparatus

3 TEST RESULTS AND DISCUSSIONS

3.1 *Elasto-Plastic Behavior*

Figure 6 shows the relations of the beam load P and the vertical deformation δ at the beam loading point. In these figures, a symbol R indicates the rotation angle of the beam loading point. The slight solid lines and the dotted and dashed lines indicate the mechanism lines. Former shows the flexural strength of the beam member $_BP_p$ and later shows the shear strength of the beam-to-column connection panel $_PP_p$.

The test results are indicated in Table 3. Rotation angles are indicated in Table 3, where the local buckling occurred at the connection panel and at the beam flange. And in this table, the rotation angles of occurrence of the crack at the welding portion of the beam flange are indicated, too.

For the specimens MP-N and MP-S, the strength did not deteriorate, up to the rotation angle R=7.0%. The maximum strength reached 1.7_PP_p. The extra strength beyond the theoretical shear strength was provided by resistance of the column flanges and stiffeners, that of strain hardening of the connection panel, and that of the beam and column web adjacent to the panel.

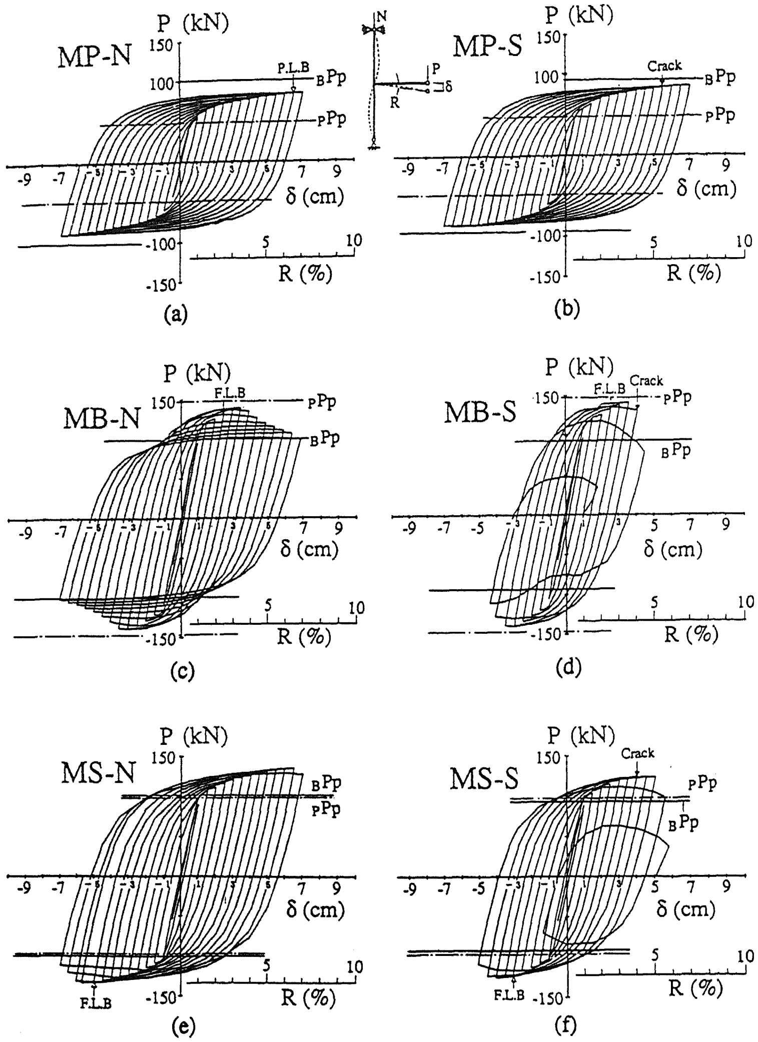
As for the specimens MB-S and MS-S with the scallops at the beam end, the crack occurred at the welding portion of the beam flange and finally the beam flange fractured at the heat affected zone (see Photo.1). On the other hand, in case of the specimens MB-N and MS-N, the crack did not occur.

3.2 *Effect of Collapse Modes on Behavior of Frames with non-scallop*

Figure 7 (a) shows the relation of the beam load $P/_BP_p$ and the accumulated deformation $\Sigma\delta/\delta_p$ of the specimen MS-N, where δ_p is the theoretical elastic displacement at $_BP_p$. Figures 7 (b) and (c) show the envelope curves of the relations of $P/_BP_p$-$\Sigma\delta/\delta_p$' of the specimens without the scallops and with the scallops, respectively.

In this paper, the deformation capacity is defined as the deformation where the strength of each specimen reaches the maximum strength.

The specimen MP-N shows large deformation capacity,



F.L.B: Local Buckling of Beam Flange
P.L.B: Loacl Buckling of Connection Panel
Crack : Crack at Beam Flange

Fig.6 Beam Load Deflection Relations

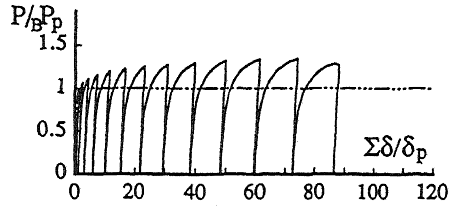
but, as for the stiffness, the strength and the energy-dissipation capacity, this mode is inferior to other collapse modes.

The specimen MB-N shows the smallest deformation capacity. Figure 8 shows the relations between the beam load P and the shear deformation angle γ of the beam-to-column connection panel. As for this specimen, the shear deformation angle of the connection panel is the smallest in the three modes. From this result, it is clear that the rate of the flexural deformation of the beam member to the total deformation of the frame is the largest of the three modes. So, the local buckling of the beam flange occurred at earlier stage.

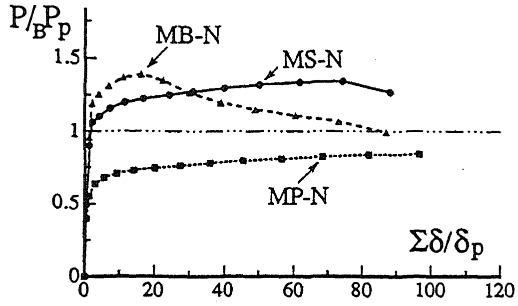
The specimen MS-N shows large ductility and large energy-dissipation capacity.

3.3 *Effect of Presence of Scallops on Behavior of Frames*

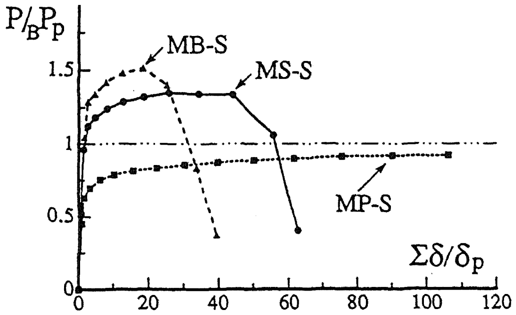
As for the Mode P, the influence of the presence of the scallops on the behavior of the frames is little. As for the Mode B and Mode S; in case of the specimens with the scallop at the beam end, finally the beam flange fractured at the heat affected zone, and then the strength deteriorated suddenly. On the other hand, for specimens with non scallop, the crack did not occur and the strength deteriorated slowly because of occurrence of the local buckling of beam flange.



(a) MS-N



(b) Specimens with Non Scallop



(c) Specimens with Scallops

Fig.7 P/$_B$Pp - Σδ/δp Relations

3.4 *Energy Dissipation Capacity*

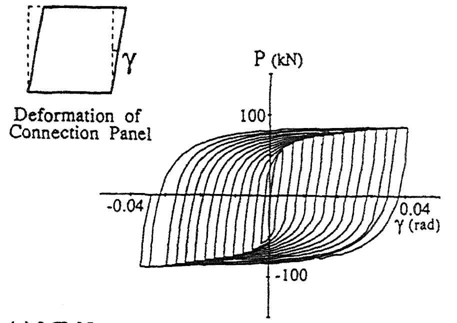
Figure 9 shows the relations of the energy dissipation capacity E and the rotation angle R. In case of the specimens without the scallops, finally the frame of the Mode S shows the largest energy dissipation capacity. On the other hand, in case of the specimens with the scallops, specimens MS-S and MB-S show small capacity, because the beam flange fractures at early stage.

3.5 *Ductility of Frames*

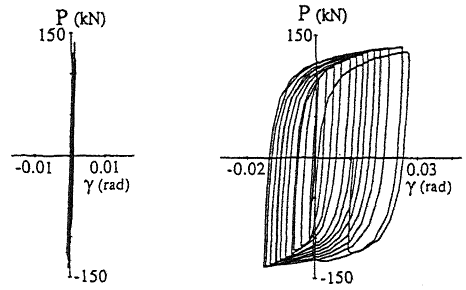
The equivalent ductility factor μ is defined as

$$\mu = \frac{\delta cr}{\delta p} \tag{2}$$

where δcr is the deflection in which the area of the dark

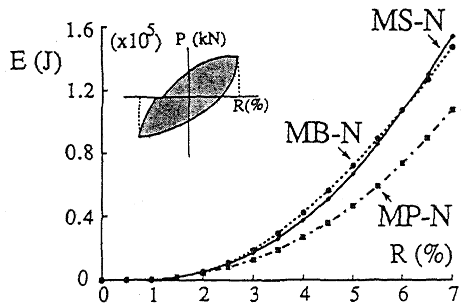


(a) MP-N

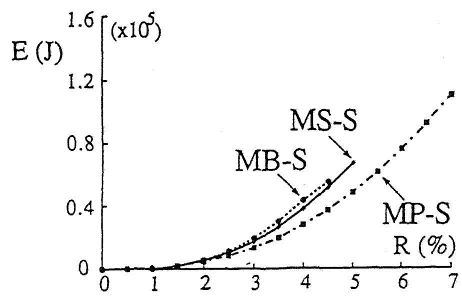


(b) MB-N (c) MS-N

Fig.8 P - γ Relations



(a) Specimens with Non Scallop



(b) Specimens with Scallops

Fig.9 Energy Dissipation Capacity

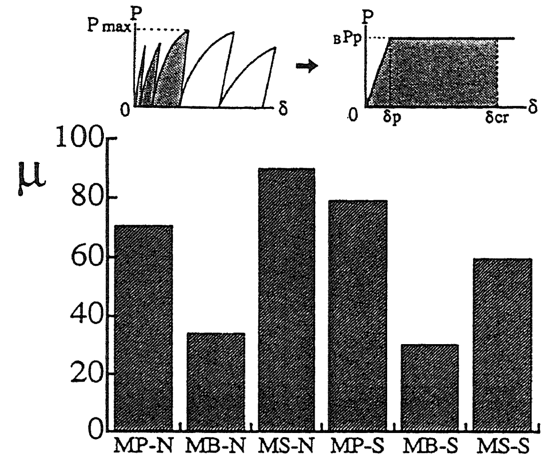


Fig.10 Equivalent Ductility Factor μ

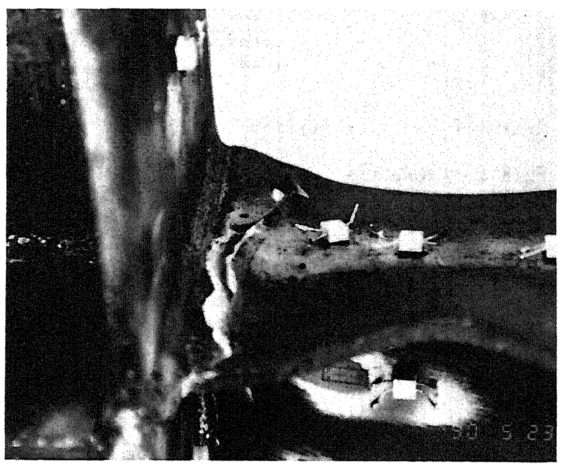


Photo.1 Fracture of Beam Flange

shading on ideal P-δ relation equals to the accumulated energy, dissipated up to the stage that the beam load P attain the maximum strength (see Fig.10). The values of μ for all specimens are shown in Fig.10 and Table 3. Figure 10 shows that the specimen MS-N behaves the most ductile of all specimens. The ductility of the specimen MB-N results in a third of that of the specimen MS-N, in consequence of reinforcing the beam-to-column connection panel excessively.

4 CONCLUSIONS

It has become clear from the test results that;

1)As for the effect of the collapse modes on the behavior of steel frames with non-scallop;

1-1)The frame of the Mode S where both beam and beam-to-column connection panel collapse simultaneously in flexure and in shear respectively, possesses the best earthquake resistant properties among the frames of three collapse modes: the largest energy dissipation capacity and the largest ductility.

1-2)The frame of the Mode P where connection panel collapses in shear, shows large deformation capacity, but as for the energy dissipation capacity this mode is inferior to the frame of the Mode S.

1-3)The frame of the Mode B where beam collapses in flexure, shows large energy dissipation capacity, but as for the ductility this mode is inferior to the frame of the Mode S.

2)As for the effect of the presence of the scallops on the behavior of steel frames with the Mode B and Mode S;

2-1)The effects of the presence of the scallops on the ductility and energy dissipation capacity, are large.

2-2)In case of the frames with scallops at the beam end, the crack occurs at the welding portion of the beam flange. Finally the beam fractures at the heat affected zone and then the strength deteriorates rapidly.

2-3)In case of the specimens with non scallop, the crack does not occur and the strength deteriorates slowly because of occurrence of the local buckling of the beam flange.

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