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EFFECT OF COLLAPSE MODES ON EARTHQUAKE RESISTANT PROPERTIES FOR STEEL FRAMES

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ABSTRACT

Effect of collapse modes of steel frames on earthquake resistant properties was investigated; strength, ductility and energy dissipation capacity. The authors divided collapse modes of steel frames roughly into four classes; column collapse mode, beam collapse mode, beam-to-column connection panel collapse mode and both beam and connection panel yield collapse mode. In this paper it makes clear how collapse mode is the best on the earthquake resistant properties, and how much the values of strength ratio between members and connection panels are recommended in order to realize the best collapse mode on the earthquake resistant properties. But the yield strength of steel members scatter generally, so the collapse mode can not be controlled easily. In this paper the effect is discussed on a reliability analysis: difference of the variation coefficient of the yield strength of members and one of the correlation between the yield strength for beams, columns and connection panels on the collapse modes of steel frames and on the earthquake resistant properties.

KEYWORDS

Montecalro simulation; steel frames; collapse modes; scatter of yield strength; energy dissipation capacity; ductility

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 buckling of members and mechanical properties of steel. Behavior of members is affected by the local bucklings, however the behavior of beam-to-column connection panel shows stable and strength degradation is scarcely observed in the connection even if diagonal buckling occurs in the panel owing to the shear force (Naka, Kato, et al., 1969).

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 do not always show large ductility and large energy dissipation capacity. Strength, ductility and energy dissipation capacity of steel frames are affected by collapse modes of frames. The authors published that frames designed to make beam members and connection panels yield in flexure and in shear respectively show larger ductility and larger energy dissipation capacity (Both Beam and Panel Yield Mode) than ones designed to make only beam members yield in flexure and to make only connection panel yield in shear (Matsui and Sakai 1992). The above state is based on the reason that flexural strength of members deteriorates by local buckling after displacement exceeds the value corresponding to maximum strength, on the other hand shear strength of beam-to-column connection panels do not deteriorate and show large ductility and large energy dissipation capacity even if local buckling occur at the panel.

But the yield strength of steel members scatter generally, so collapse modes of frames can not be controlled easily. In this paper it makes clear on a reliability analysis how much the value of ratio of the shear strength of beam- to-column connection panels to the flexural strength of members is recommended to make frames with Both Beam and Panel Yield Mode. And the following effect is discussed : difference of variation coefficient of the yield strength and coefficient of correlation between the yield strength for beams, columns and connection panels on the collapse modes and on the earthquake resistant properties.

OUTLINE OF ANALYSIS

Montecalro simulation was performed that ; one thousand random yield strength obeyed the normal distribution were generated for beams, columns and connection panels each other. The nominal value of the yield strength, the mean value of them and the variation coefficient of them were quoted from (AIJ, 1990). Then by using the random yield strength, one thousand relation curves of Horizontal Load and relative story displacement of steel frames were calculated on elasto-plastic analysis.

Table 1 Statistic of steel yield strength (AIJ, 1990)

Standard (JIS)	Nominal Yield Strength (ton / cm^2)	Mean Value of Yield Strength (ton / cm^2)	Standard Deviation (ton / cm^2)	Variation Coefficient (%)
SS400	2.4	2.79	0.363	13.0

Analytical Model and Parameters

An analytical model was a cruciform steel frame under earthquake loading as shown in Fig. 1. H- shaped steels were used for beams and columns. Parameters were selected as follows,

1) α : ratio of the shear strength of the beam-to-column connection panel to the flexural strength of the beam member, calculated by using the nominal yield strength (= from 0.5 to 1.5 by 0.1 step).

$$\alpha = {}_pR_p \,/\, {}_bR_p \tag{1}$$

where ${}_pR_p$ = reaction at the beam end when beam-to-column connection panel yields in shear.
${}_bR_p$ = reaction at the beam end when the moment of the beam at the column face equals to the full plastic moment.

2) β : ratio of the nominal flexural strength of the column member to one of the beam (= 1.1, 1.3, 1.5) ,

$$\beta = {}_cR_p \,/\, {}_bR_p \tag{2}$$

where ${}_cR_p$ = reaction at the beam end when the moment of the column at the beam face equals to the full plastic moment.

3) V : value of variation coefficient of yield strength for steel members,

4) ρ : value of coefficient of correlation between the yield strength for beam members, one for column members and one for connection panels.

The axial force for the column member is 30 % of the nominal axial yield strength of the cross section. Values of α were controlled by changing the thickness of connection panels, considering reinforcing by doubler plates, and β was controlled by changing the width and the thickness of flanges of column members. The size of beam and column members are shown in Table 2.

Table 2 Size of members

Member	β	Size
Beam	-	BH - 200 x 100 x 4.5 x 9
	1.1	BH - 200 x 136 x 4.5 x 9
Column	1.3	BH - 200 x 170 x 4.5 x 9
	1.5	BH - 200 x 157 x 4.5 x 12

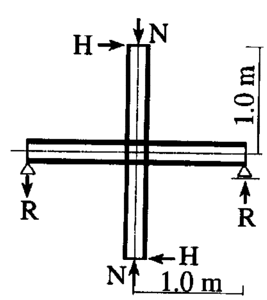
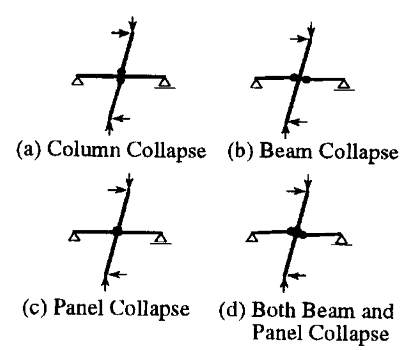


Fig. 1 Analytical Model



Relation of Panel Moment and Shear Deformation Angle for Connection Panel

A beam-to-column connection panel was taken apart to a web panel and a surrounding frame composed of column flanges and beam ones. And connection panel moment of the connection panel was given by summing moments of each parts. It is assumed that the shear stress is uniformly distributed over the web panel and the column axial force is carried by the column flanges. So the yield shear strength of the web panel $_pM_p$ and the yield shear deformation $_p\gamma_p$ are given by Equations (3) and (4).

$$_pM_p = \frac{\sigma_y \cdot V_p}{\sqrt{3}}, \qquad _p\gamma_p = \frac{_pM_p}{G \cdot V_p} \tag{3), (4}$$

$$V_p = (_bD - {_bt_f}) \cdot (_cD - {_ct_f}) \cdot t_p \tag{5}$$

And the plastic moment of the surrounding frame $_fM_p$ and the deformation $_f\gamma_p$ are given by Equations (6) and (7).

$$_fM_p = 4 \cdot \min\,(_{cf}M_p,\ _{bf}M_p) \tag{6}$$

$$_f\gamma_p = \frac{_{bf}I \cdot (_bD - {_bt_f}) + {_{cf}I} \cdot (_cD - {_ct_f})}{24 \cdot E \cdot {_{bf}I} \cdot {_{cf}I}} \tag{7}$$

where E and G = Young's modulus and shear modulus, respectively
$_bD$ and $_cD$ = Depth of beam and column, respectively
$_bt_f$ and $_ct_f$ = Thickness of beam flange and column flange, respectively
t_p = Thickness of connection panel
$_{bf}M_p$ and $_{cf}M_p$ = Full plastic moment of beam flange and column flange, respectively
$_{bf}I$ and $_{cf}I$ = Moment of second order of beam flange and column flange, respectively

It is assumed that the plastic rigidity of the web panel and one of the surrounding frame are also 0.02 time as much as the elastic ones, considering the test results (Matsui and Sakai 1992). From the above mentioned and considering the value of $_f\gamma_p$ is much larger than one of $_p\gamma_p$, the relationship of the connection panel moment and the shear deformation of the connection panel is given by the following equations.

$$_pM = G \cdot V_p \cdot \gamma + \frac{_fM_p}{_f\gamma_p} \cdot \gamma \qquad \text{when } 0 \le \gamma \le {_p\gamma_p} \tag{8}$$

$$_pM = {_pM_p} + \frac{2}{100} \cdot G \cdot V_p \cdot (\gamma - {_p\gamma_p}) + \frac{_fM_p}{_f\gamma_p} \cdot \gamma \qquad \text{when } {_p\gamma_p} < \gamma \le {_f\gamma_p} \tag{9}$$

$$_pM = {_pM_p} + \frac{2}{100} \cdot G \cdot V_p \cdot (\gamma - {_p\gamma_p}) + {_fM_p} + \frac{2}{100} \cdot \frac{_fM_p}{_f\gamma_p} \cdot (\gamma - {_f\gamma_p}) \qquad \text{when } {_f\gamma_p} < \gamma \tag{10}$$

Relation of Moment and Rotation Angle for Members

Relations of moment and deformation of members were calculated by using relationships of moment M and curvature ϕ, and by using an assumption. M-ϕ relations were calculated on the assumption that plane sections remain plane. The stress - strain relationships of steel plates are bilinear model whose plastic modulus is 0.01 times as much as the elastic one. Rotation angle θ of members is given by the following equation in elastic range and the same equation is assumed to hold in plastic also.

$$\theta = \phi \cdot L / 3 \tag{11}$$

where L = Length of members

Specific Forms of Collapse Modes of Steel Frames

The Relationships between horizontal load H and displacement δ at the column top for the cruciform frame (see Fig. 1) were calculated by load incremental analysis, until one of beam, column and connection panel reached their own limit deformation at first. The ductility of frames was defined as the maximum displacement at the column top. The limit deformations θ_{max} for beams and columns, and γ_{max} for connection panels are as follows. Here equation (12) is quoted by (AIJ, 1990) and based on deteriorating strength of members by the local buckling, and Equation (13) is based on that the shear strength of the connection panel does not deteriorate even if the local buckling occur at the panel.

$$\theta_{max} = 5 \cdot \theta_p \qquad (\textit{for beams and columns}) \qquad (12)$$
$$\gamma_{max} = 20 \cdot {}_p\gamma_p \qquad (\textit{for connection panels}) \qquad (13)$$

where θ_p = Rotation angle of members whose face moment reaches the full plastic moment

Collapse modes of the frames were classified into four groups as follows,
- Column Collapse Mode : Only column reaches the limit deformation and the others do not reach limits.
- Beam Collapse Mode : Beam reaches the limit deformation and panel in elastic.
- Panel Collapse Mode : Panel reaches the limit deformation and beam in elastic.
- Both Beam and Panel Yield Mode : Beam reaches the limit deformation and panel in plastic, or panel reaches the limit deformation and the beam in plastic.

RESULTS OF ANALYSIS AND DISCUSSIONS

Relationship of Load and Deformation of Steel Frames Calculated with Nominal Yield Strength

Fig. 2 shows relationships of horizontal load H and horizontal deformation δ of steel frames at the column top calculated with the nominal yield strength (σy =2.4 ton/cm^2; σy = yield strength of steel plates). From the analytical results with changing the value of α which is the ratio of shear strength of connection panel to the flexural strength of the beam, the collapse modes resulted shown in Table 3. The frame with Both Beam and Panel Yield Mode show larger energy dissipation capacity than the frames with Beam Collapse Modes and ones with Panel Collapse Modes. Especially the frame with α equal 1.0 shows the largest ductility and largest energy dissipation capacity. The frames with Beam Collapse Mode show small ductility. The frames with Panel Collapse Mode show large deformation capacity, however the frames of this type are inferior to frames with Both Beam and Panel Collapse Modes as for the strength and energy dissipation capacity.

Table 3. Collapse mode and ratio of strength of connection panel to one of beam

α	0.5 - 0.8	0.9 - 1.3	1.4 - 1.5
Collapse Types	Panel	Both Beam and Panel	Beam

α: The ratio of the shear strength of a connection panel to the flexural strength of a beam

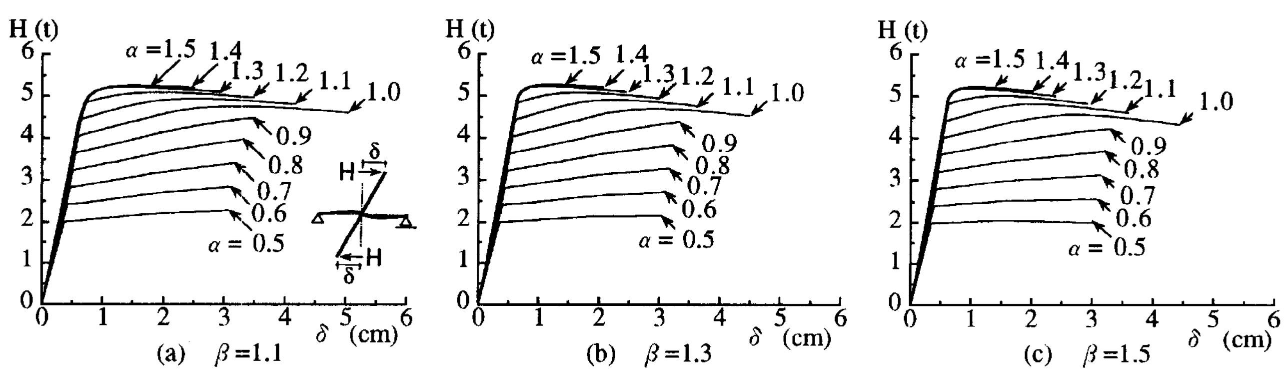


Fig. 2 Relationships of horizontal load and horizontal deformation of steel frames

Analytical Results of Steel Frames composed of members with independent random yield strength

Montecalro simulation was performed that: one thousand random yield strength obeyed the normal distribution were generated for beams, columns and panels each other. Then by using the random yield strength, one thousand relation curves of horizontal load H and deflection δ at the column top were calculated on elasto-plastic analysis in each α and β (α,β : see *Analytical Model and Parameters*). Frequency of collapse types obtained by the simulations are shown in Fig. 3. Frequency of frames with Both Beam and Panel Yield Mode is highest in α = 1.1 for all cases, however probability of occurring this collapse mode is 70 % at the most. If the value of β equals 1.1 and α is over 1.0, probability of occurring frames with Column Collapse Mode is about 20%. If the value of β is designed 1.5, probability of column collapse frame is kept 4% at the most.

Fig. 4 shows the mean value and variation coefficient of energy dissipation capacity. Energy dissipation capacity for the frames were to make non dimensional value by the elastic energy $_eU_{bn}$, where $_eU_{bn}$ indicates the energy dissipated until the beam moment at the column face reached the full plastic moment calculated by using nominal yield strength. For all of β , the mean of energy is the largest in α =1.0, however the variation coefficient is from 0.35 to 0.4 and scatter is wide. The scatter of energy is largest in β =1.1, because frequency of frames with Column Collapse Mode is large in β = 1.1.

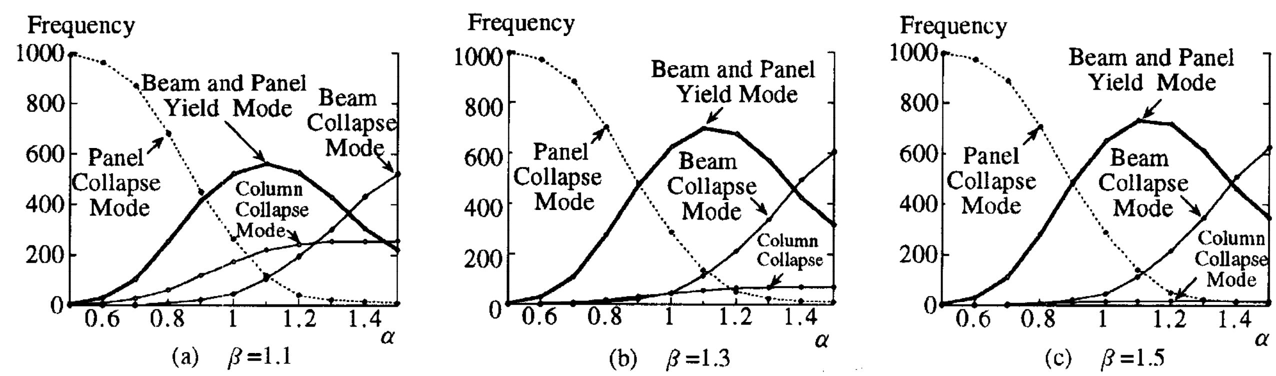


Fig. 3 Frequency of collapse modes of frames with setting V=13% and ρ = 0

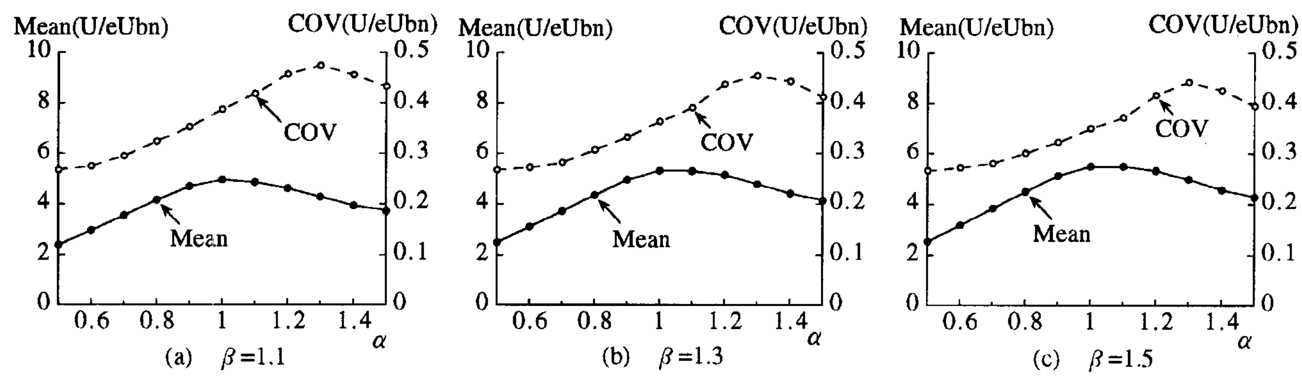


Fig. 4 Energy dissipation capacity of frames with setting$V$=13% and ρ = 0

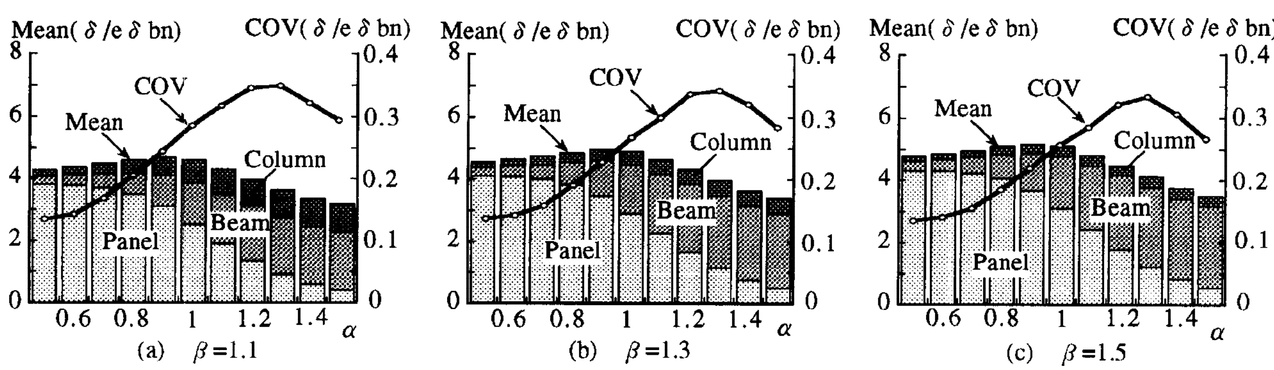


Fig. 5 Ductility of frames with setting V=13% and ρ = 0

Fig. 5 shows the ductility of the frames. The ductility was to make non dimensional value by the deflection ${}_e\delta_{bn}$, where ${}_e\delta_{bn}$ indicates the elastic horizontal deflection at the column top when the beam moment at the column face reached the full plastic moment calculated by using nominal yield strength. In these figures, the mean values of ductility are shown by height of columns, and also deflection ratios are shown for beams, columns and connection panels. The mean value of ductility is the largest in $\alpha = 0.9$ for all of β. If the value of α equals 0.5, the variation coefficient of ductility is 13%, and the value equals the variation coefficient of yield strength of steel plates. And as the value of α becomes large, the variation coefficient of ductility is large until α equals 1.3. The ratio of the deflection by shear deformation of connection panels to total deflection is over half in α = from 0.5 to 1.0.

Effect of Correlation between Yield Strength of Members on Earthquake Resistant Properties

Effect of correlation coefficient ρ between the yield strength for beams, columns and connection panels on the collapse mode and on the earthquake resistant properties were discussed. Frequency of collapse types obtained by the simulations setting $\rho = 0.8$ and 0.9 are shown in Fig. 6. Though the collapse types of frames can not be controlled in $\rho = 0$ (Yield strength of members are independent), the frames with Both Beam and Panel Collapse Modes almost can be realized by setting $\alpha =$ from 1.0 to 1.2, in $\rho = 0.8$, especially $\rho = 0.9$. The frequency of frames with Column Collapse Mode are low, as the value of ρ is large. Probability of occurrence of frames with Column Collapse Mode is kept below 3% and 1.5% by setting $\beta = 1.3$ and 1.5, respectively, however in $\beta = 1.1$ the probability is large, even if ρ setting 0.9. Fig. 7 shows the mean value and the variation coefficient of energy dissipation capacity in $\rho = 0.9$. The mean value of the energy for the frames with setting $\rho = 0.9$ is much larger and the variation coefficient is much smaller than ones with setting $\rho = 0$. This is based on that the frequency of frames with Column Collapse Mode is smaller, as the value of ρ is large. Fig. 8 shows the ductility of frames setting $\rho = 0.9$. The mean value of the ductility is largest in $\alpha = 1.0$ for each cases. The variation coefficient of the ductility is constantly 13% in α being form 0.5 to 0.8. This is based on that almost of frames collapse at connection panel in this region. Fig. 9 shows the difference of the value of ρ on the energy dissipation capacity and on the ductility for the frames with setting $\alpha = 1.0$ and $\beta = 1.3$. As the value of ρ become large, the mean value of the energy and the ductility is

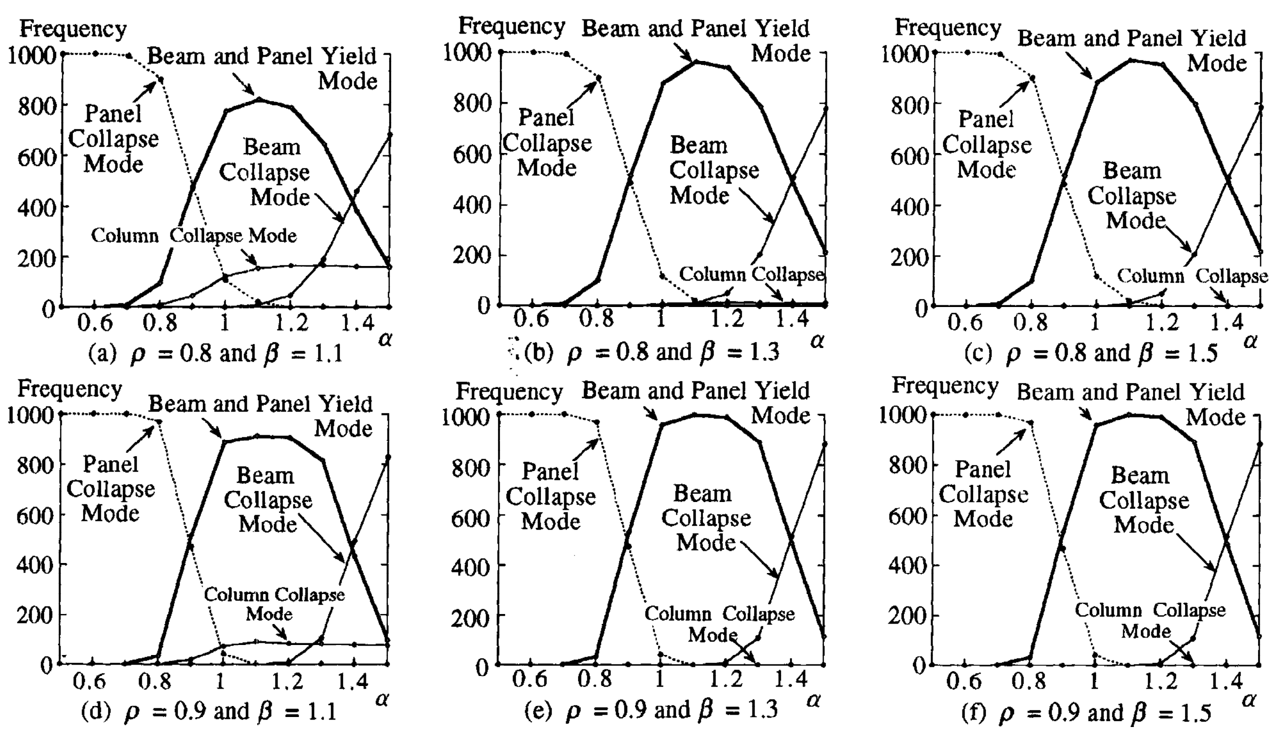


Fig. 6 Frequency of collapse modes of frames with setting V=13% and ρ = 0.8, 0.9

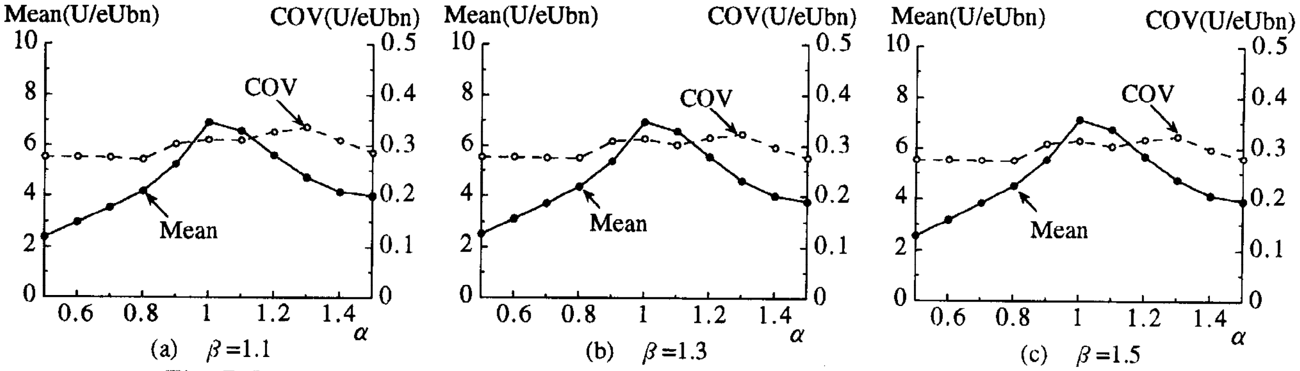


Fig. 7 Energy dissipation capacity of frames with setting V =13% and ρ = 0.9

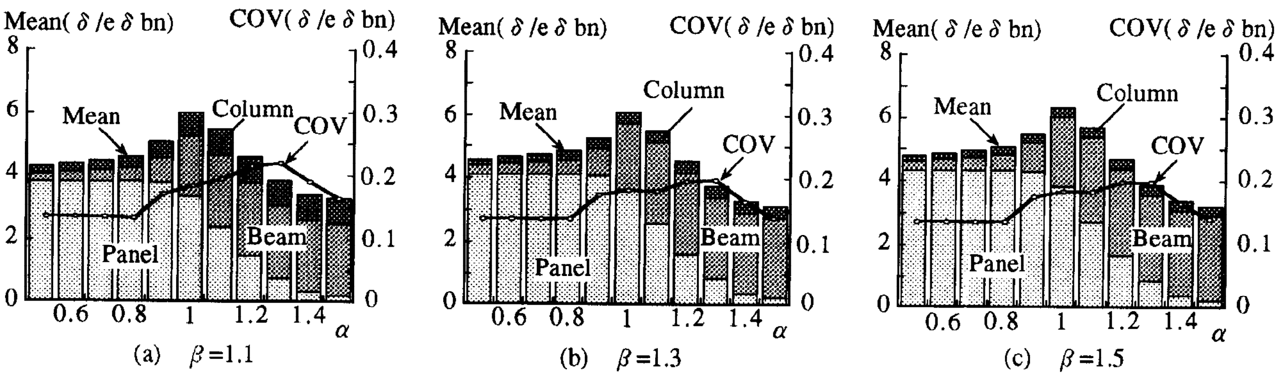


Fig. 8 Ductility of frames with setting V =13% and ρ = 0.9

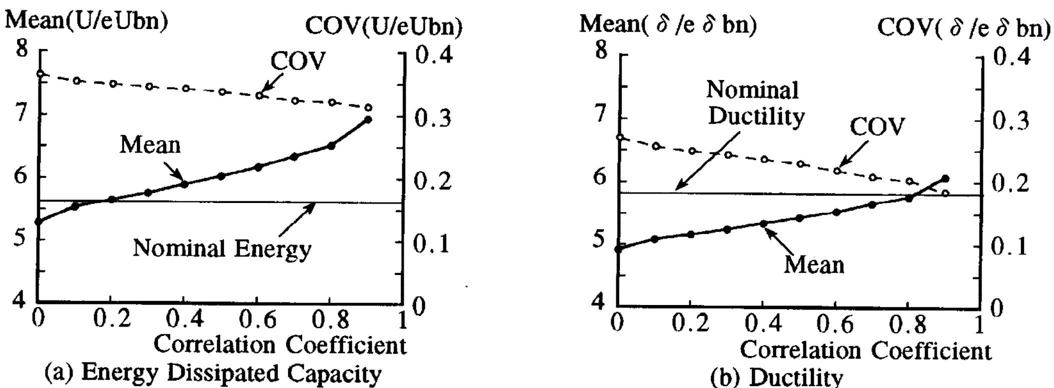


Fig. 9 Effect of correlation coefficient between yield strength of members on energy and on ductility

large and the value of the variation coefficient of them is small. The mean value of the ductility in ρ = 0.9 exceeds the nominal ductility which is calculated by using nominal yield strength, however in ρ being lower 0.8, the mean value of ductility is below the nominal ductility.

Effect of Variation Coefficient of Yield Strength of Members on Earthquake Resistant Properties

Effect of variation coefficient V of the yield strength on the collapse modes and on the earthquake resistant properties was studied. Fig. 10 shows the results of the frames with setting V = 5%, β = 1.3 and ρ = 0. From the results, it is clear that if the value of V is 5%, the frames with Both Beam and Panel Yield Mode can be realized over 95% by setting α from 1.0 to 1.2. The frequency of frames with Column Collapse Mode is to make almost zero. Fig. 10 (b) and (c) shows the energy and the ductility in V = 5%, in respectively. From the results, The mean value of the energy and one of the ductility for the frames with setting V = 5% and ρ = 0 are almost same to ones with setting ρ = 0.9 and V = 13%, however the variation coefficient of the former frames is much smaller than one of the latter frames. Fig. 11 shows the effect of variation coefficient of the yield strength on the energy and the ductility for the frames with setting α =1.0, β = 1.3 and ρ = 0. From the results, as the value of the variation coefficient of the yield strength become small, the mean

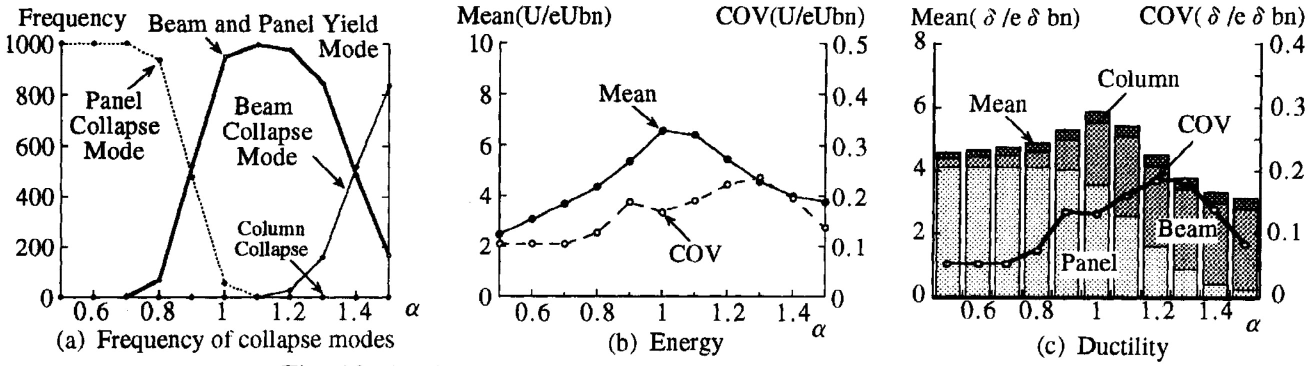


Fig. 10 Analytical Results of frames with setting V =5% and ρ = 0

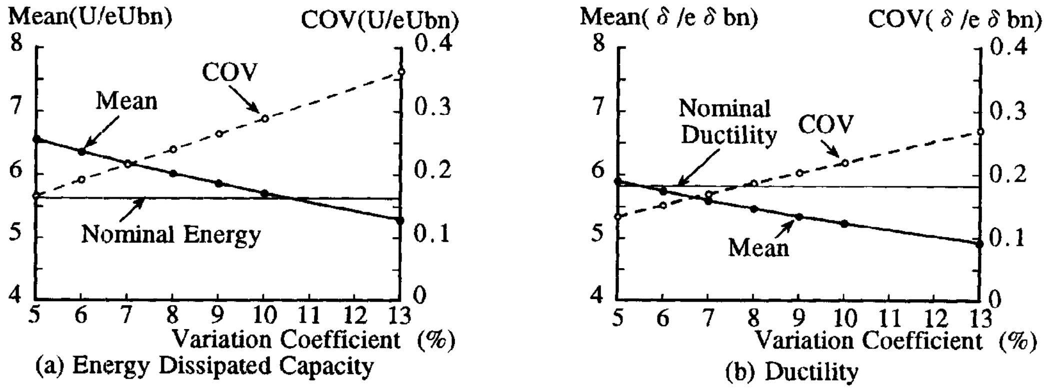


Fig. 11 Effect of variation coefficient of yield strength on energy and ductility

values of the energy and one of the ductility for frames are large and the variation coefficient of them are small. The mean value of the ductility in $V = 0.5$ exceeds the nominal ductility which is calculated by using nominal yield strength, however in V being upper 0.5, the mean value of the ductility is below the nominal ductility.

CONCLUSIONS

It has become clear from the analytical results that:

1) Frames yielding at both beam and beam-to-column connection panel in flexure and in shear respectively possess good earthquake resistant properties; large energy dissipation capacity and ductility.
2) If the value of variation coefficient of yield strength of members is about 13% and yield strength of members and one of connection panel are independent, collapse modes of frames can not be controlled and scatter of the earthquake resistant properties is large.
3) Collapse modes can be controlled by setting coefficient of correlation 0.9 between yield strength of members and connection panel, or by setting variation coefficient of yield strength below 5%. In order to make frames with Both Beam and Panel Yield Mode, the ratio of the shear strength of connection panels to the flexural strength of beams should be set from 1.0 to 1.2.
4) In order to make scatter in energy dissipation capacity and ductility lower, the coefficient of variation of yield strength of steel members should be restricted below 5%.

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