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The effect of weak beam-to-column joint panels on seismic performance of steel building structures

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ABSTRACT: The effect of weak beam-to-column joint panels on seismic performance of steel building structures was discussed in this paper. The key factor of this study is the strength of beam-to-column joint panels. The dynamic time-history response analysis based on the energy concept was carried out in order to obtain an optimum strength of joint panels under the both severe earthquakes and medium earthquakes. The results from the analyses showed that weak joint panel structures were most preferable to preclude damage concentration in a particular story, and had effect to decrease the design yield shear force of structures.

1 INTRODUCTION

In high seismic zones such as almost all areas of Japan, it is recommended that the beam-to-column joint panels should be strengthened to remain elastic under seismic design loads representing medium earthquake motions. To keep this requirement, strengthening of beam-to-column joint panels with cover plates or stiffeners is often needed. However, such strengthening as with cover plates or stiffeners usually requires difficult and tedious design work of details, and further, the efficiency of the strengthening is sometimes doubtful.

On the other hand, when a moment-resisting steel structure has no treatment for strengthening joint panels in the allowable stress design, the panel yield strength is generally less than the lesser yield strength of beams and columns. Thus, the seismic failure mode of this type of structures may be determined by the yielding of joint panel prior to the yielding of the adjacent members; this type of failure is hereinafter called "joint panel failure mode". According to a lot of test data, however, beam-to-column joint panels have generally a large capacity of energy absorption owing to stable shear deformation. This capacity may be used effectively to preclude a moment-resisting steel structure from collapse under severe earthquakes.

At the present time, the "joint panel failure mode" is not yet recognized to be recommendable since the influence of joint panel yielding on the overall seismic behavior has not yet fully understood. Here, there arises a significant question whether weak joint-panel structures is preferable or not in seismic design against severe earthquakes. This paper tried to answer the above question with appropriate structural modeling and by the seismic energy concept.

The primary objective of this paper is then to investigate the influence of strength of beam-to-column joint panels on the overall seismic behavior in steel structures and to propose an optimum strength of joint panels. To achieve the object, the following 4 analyses(A,B,C,and D) were conducted.

A) The influence of the failure mechanism on total energy input into the structures (column failure mode, beam failure mode, and joint panel failure mode)

B) Relationships between joint panel strength and n-value which indicates the extent of damage concentration.

C) The effect of energy absorption of joint panel on reduction in design yield shear force coefficient of first story in structures.

D) The seismic performance of weak joint panel structures in terms of the maximum interstory drift under medium earthquakes.

2 ANALYTICAL MODELING AND METHODS

The analyses were carried out through the dynamic time-history response analysis. The analytical model is five-story flexural frames including shear deformation of beam-to-column joint panels. The basic dynamic system used for the analysis is a single column model having masses concentrated into the center of beam-to-column joint panels as shown in Fig. 1. It is assumed that each particle of mass has the quantity of m.

Inelastic properties of the members, plastic hinges are assumed to be formed at the ends of beams and columns. Furthermore, at the joint panels, elastoplastic rotation hinges representing shear deformation of the panels are considered. The restoring force characteristics of these inelastic elements are assumed to have elastoplastic bi-linear

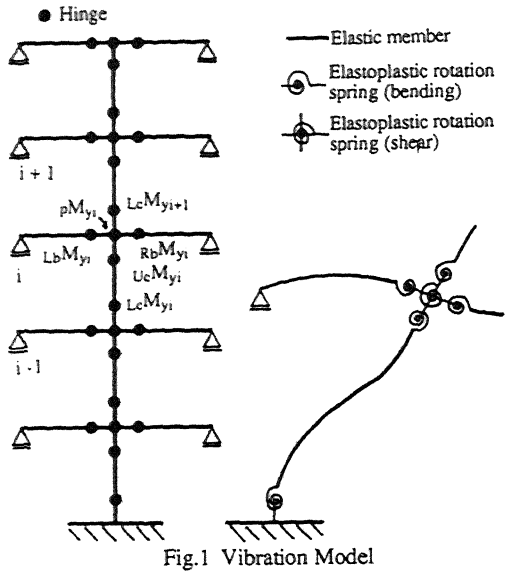


Fig.1 Vibration Model

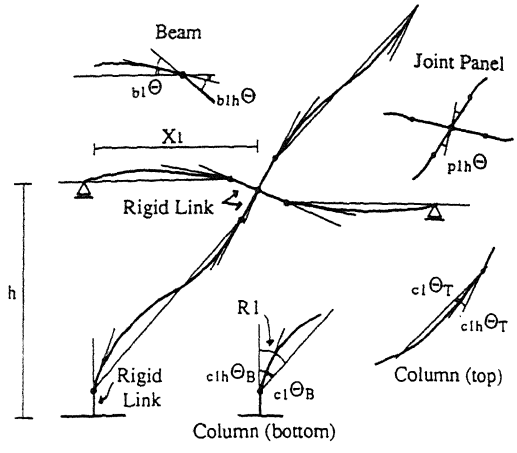


Fig.2 Compatibility Condition of Model

relationships. The freedom of each story is only translational deformation in one direction, which were transformed from the rotational deformation of each member as shown in fig.2.

It also depicts the compatibility condition between interstory drift and rotation angle of each member. In this model, interstory drift angle (Ri) of i-th story is expressed by the following formulas.

$$R_i = \frac{X_i}{h} = {}_{bi-1}\Theta + {}_{bi-1h}\Theta - {}_{ci}\Theta_B - {}_{cih}\Theta_B + {}_{pi-1h}\Theta$$

(at the bottom of i-th story)

$$R_i = \frac{X_i}{h} = {}_{bi}\Theta + {}_{bih}\Theta - {}_{ci}\Theta_T - {}_{cih}\Theta_T + {}_{pih}\Theta \qquad (1)$$

(at the top of i-th story)

where, Xi is interstory drift of i-th story, h is story height, $bi\theta$ is rotation angle of elastic beam of i-th story, $bih\theta$ is hinge rotation angle of beam, subscript c and p express the member of column and joint panel respectively, subscript T and B indicate the location at the top of column and at the bottom of column respectively. Length of rigid links were assumed to be 0 in the model.

In the analysis, the major parameter is the panel yield strength, which is represented by Rpy. Rpy is defined as follows.(see Fig.1)

$$R_{py} = {}_pM_{yi} / \min\left[({}_{Lb}M_{yi} + {}_{Rb}M_{yi}),({}_{Uc}M_{yi} + {}_{Lc}M_{yi+1})\right] \qquad (2)$$

where min[A,B] means the lesser of A and B, and where LbMyi and RbMyi are yield moments of left and right hand side beams adjacent to the joint panel, and UcMyi and LcMyi+1 are yield moment of the upper and lower columns adjacent to the joint panel.

As input waves, the two recorded ground motions, Hachinohe Earthquake 1968(EW, max.1.83m/sec.^{2}) and El Centro Earthquake 1940(NS, max. 3.10m/sec.^{2}) were chosen for the analysis.

3 ANALYTICAL RESULTS AND DISCUSSIONS

3.1 Total energy input

The Analysis (A) was carried out to investigate the influence of failure mechanisms on total energy input into the model. Three different failure modes, that is, column failure mode, beam failure mode and joint panel failure mode were independently implemented in the analysis. Namely, to make one particular failure mode predominant, plasticization of inelastic elements concerned with other failure modes was not allowed in setting the model.

In this model, yield shear force coefficient of first story($\alpha 1$) was set to be 0.2.

$$\alpha_i = \frac{Q_{yi}}{\sum_{K=i}^{N} m_k g} \qquad (3)$$

where Qyi; yield shear force of i-th story
m; mass of k-th story
g; acceleration of gravity

Fig. 3 shows the result of the Analysis. Total energy input into the models was converted to a psuedo-velocity V_E which is defined as (Akiyama 1985)

$$V_E = \sqrt{\frac{2E}{M}} \qquad (4)$$

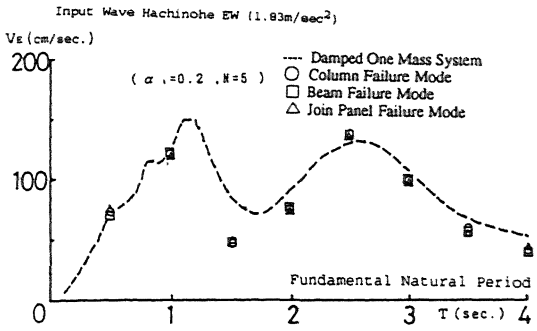


Fig.3 Total Energy Input

where E is the total energy input, M is total mass.

From this figure, it can be seen that the total energy input represented by V_E depends primarily on the fundamental natural period T of the system, and barely on the predominant failure modes. Also in this figure, the dashed line designates the V_E values obtained from a damped one-mass system with h = 0.1. It is then found that the plotted values on the five-mass system are close to the dashed line.

3.2 Damage concentration

The purpose of the Analysis (B) is to make clear the relation between the joint panel strength and the damage concentration into the structure in which the yielding of joint panels is ahead of the yielding of beams and columns. In this analysis, the values of Rpy were varied from 0.3 to 1.3. The ratios of beam strength to column strength were varied from 1.0/1.4 to 1.4 considering realistic combinations of beams and columns.

In this model, yield shear force coefficient of first story ($\alpha 1'$) was set to be 0.2. Here, definition of $\alpha i'$ is not the same as that of αi(Eq.3). $\alpha i'$ is defined as

$$\alpha_i' = \frac{Q_{yi'}}{\sum_{K=i}^{N} m_k g} \tag{5}$$

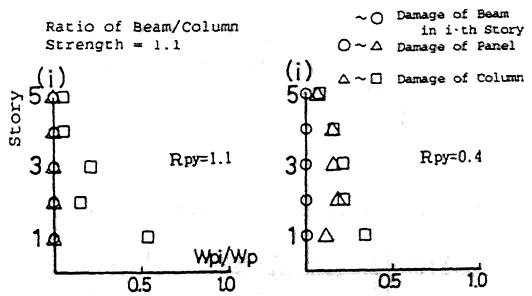


Fig. 4 Damage Distribution along the Height of Model Structures

where Qyi' is shear force of i-th story decided by the lesser yield strength of beams and columns of i-th story.

Fig. 4 shows an example of the results of the Analysis (B). In the two analyzed structures of the figure, ratio of beam strength to column strength was set to be 1.1. Furthermore, the values of Rpy were set to be 1.1 and 0.4 respectively.

The ordinate of the figure designates the story number, and the abscissa indicates the degree of the damage concentration into the i-th story, Wpi/Wp, where these symbols are defined as follows; Wpi is the absorbed energy by inelastic hysteresis into the i-th story, which is calculated by

$$W_{pi} = cW_{pi} + \frac{K_i}{K_i+K_{i+1}}({}_pW_{pi}+{}_bW_{pi}) + \frac{K_i}{K_i+K_{i-1}}({}_pW_{pi-1}+{}_bW_{pi-1}) \tag{6}$$

where Ki is the story stiffness of the i-th story, bWpi is the total energy absorbed into the right- and left-hand side beams of the i-th story, pWpi is the absorbed energy of the panel of the i-th story and cWpi is the sum of the absorbed energy at the both ends of the i-th story column, UcWpi + LcWpi. Furthermore, the total input plastic energy into a structure, Wp, is given by

$$W_p = \sum_{i=1}^{N} W_{pi} \tag{7}$$

From Fig. 4, it can be seen that the damage into the panels having the strength ratio of Rpy = 0.4 is obviously increased in each story, compared with the case of Rpy = 1.1 where there is no damage in the panels. Moreover, from this figure, we can find that in the case of Rpy = 0.4 the extent of damage concentration into the column at the first story becomes considerably less than that in the case of Rpy = 1.1; this means that the damage distribution along the height of the structure becomes uniform by decreasing the panel strength, Rpy.

3.3 Damage concentration factor

To study more clearly the peculiarity of damage concentration resulted from the change of Rpy, a new factor, damage concentration factor, n, should be introduced. This factor is included in the equation that defines the damage distribution over all stories as follows (Akiyama 1985);

$$\frac{W_{pk}}{W_p} = \frac{S_k \cdot P_k^{-n}}{\sum_{j=1}^{N} S_j \cdot P_j^{-n}} \tag{8}$$

where n = damage concentration factor, $Pj = (\alpha j/\alpha 1) / \overline{\alpha_j}$, $\overline{\alpha_j}$ = optimum yield-shear force coefficient distribution (Akiyama 1985), and

$$S_j = \left(\sum_{k=j}^{N} m_k/M \right)^2 \overline{\alpha_j^2} (K_1/K_j)$$

The damage concentration factor, n, is then obtained from Eq. 8 by using the results of response analysis where only the story shear coefficient of the k-th story, αk, is changed from the optimum one, $\overline{\alpha_k}$. That is, the damage concentration factor, n, is expressed as

$$n = - l_n \left\{ \frac{b(1-a)}{a(1-b)} \right\} / l_n Pd \qquad (9)$$

where a is given by Wpk/Wp with the optimum shear coefficient $\overline{\alpha_k}$ and b is Wpk/Wp with the changed shear coefficient, $\alpha k = Pd \cdot \overline{\alpha_k}$. The values of a and b can be concretely attained by response analyses. The factor, n, is then considered to be an index designating the damage concentration into the k-th story having a certain weakness in story shear strength. In a shear-type multi-story structure, the value of n is found to be 12, and as the flexural component is gradually incorporated into the overall structural behavior, the n-value becomes smaller than 12 (Akiyama 1988).

Now, the factor n was evaluated on the structure models having different Rpy from 0.3 to 1.3. Herein, the coefficient Pd and the story number k were chosen to be 0.8 and 3, respectively. Needless to say, the factor n would depend on Pd and k. Thus, the story number, k = 3 is determined by the result of the past study in which the number, 3, gives the most severe damage concentration in five-story models as a safety side evaluation (Akiyama 1985), and also the selection of Pd = 0.8 gives an appropriate value of n which can designate reasonably damage concentration (Akiyama 1985).

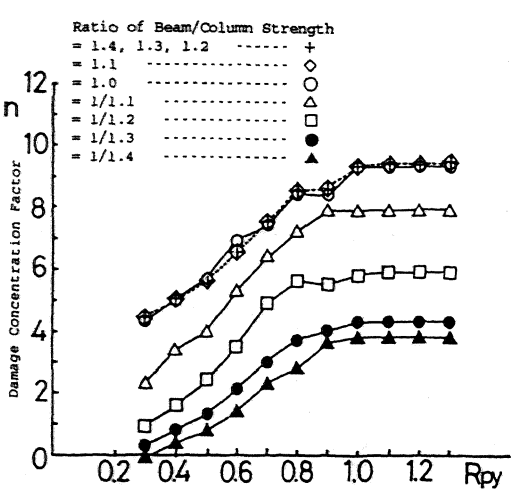


Fig. 5 Relationship between Rpy and n

Fig. 5 depicts the obtained values of n as a function of the panel strength ratio, Rpy, associated with the ratios of the beam yield strength to column yield strength. From this figure, it can be seen that in the region of Rpy<1.0, there is a tendency for damage concentration into the third story to lessen as Rpy becomes smaller.

When Rpy>1.0, on the other hand, the damage concentration does not depend on Rpy, and does depend on only the strength ratios between beams and columns; this is because the stronger joint panels do not exhibit plasticization. Thus, in this range of Rpy, the damage concentration becomes smaller as the ratio of beam strength to column strength becomes smaller.

3.4 Reduction of design yield shear coefficient

The authors (Hasegawa et al. 1991) had already demonstrated that the energy absorption mechanism produced by the panel yielding is effective to reduce the damage of beams and columns, in particular to prevent the failure of columns. The Analysis (C) was carried out to examine the effect of energy absorption of joint panel on reduction in design yield shear force coefficient of first story ($\alpha 1'$) of structures. The structural model having this $\alpha 1'$ obtained from the Analysis (C) may be effective to make optimum damage distribution to each member (beam, column, and joint panel) under sever earthquakes.

For this aim, limit of energy absorption capacity of each member are needed. Table 1 shows the limit of cumulative inelastic deformation ratio η of each member (A.I.J. 1990) used for this analysis. η is defined as follows;

$$\eta_j = \frac{{}_jW_{pi}}{{}_jM_{yi} \cdot {}_j\theta_{yi}} \qquad (10)$$

where j expresses each member of the column (c), beam (b), and joint panel (p), and where jMyi and jθyi is the yield moment and yield rotational angle of i-th story, j member.

Furthermore, in this Analysis (C), the major parameter $\alpha 1'$ was changed to be from 0.3 to 1.1, and strength ratio of beams to columns was chosen to be 1.2, and damping was assumed to be 2% for the analysis models.

Table 1 Absorption Capacity of the Member

Column	Joint Panel	Beam
$\eta_C = 12$	$\eta_P = 30$	$\eta_B = 6$

Fig.6 shows relationship between Rpy and α1'. The characters of ○, ● indicate the case in which α1' was decided by limit of energy absorption capacity of column (ηc=12). The characters of □, ■ indicate the case in which α1' was decided by that of panel (ηp=30). Fig.7 demonstrates 3 examples (1, 2, and 3 models, see Fig.6) of analytical results in terms of obtained η regarding inelastic elements in beam,

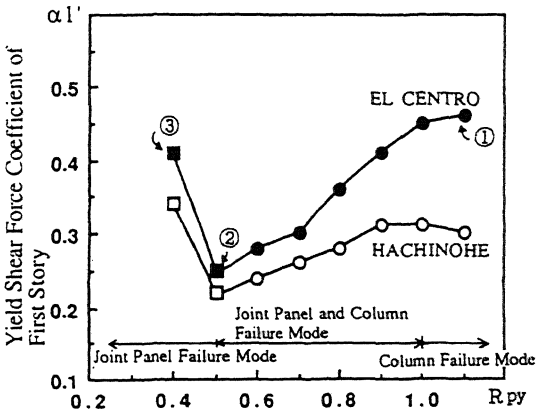


Fig. 6 Relationship between Rpy and α1'

columns and join panels.

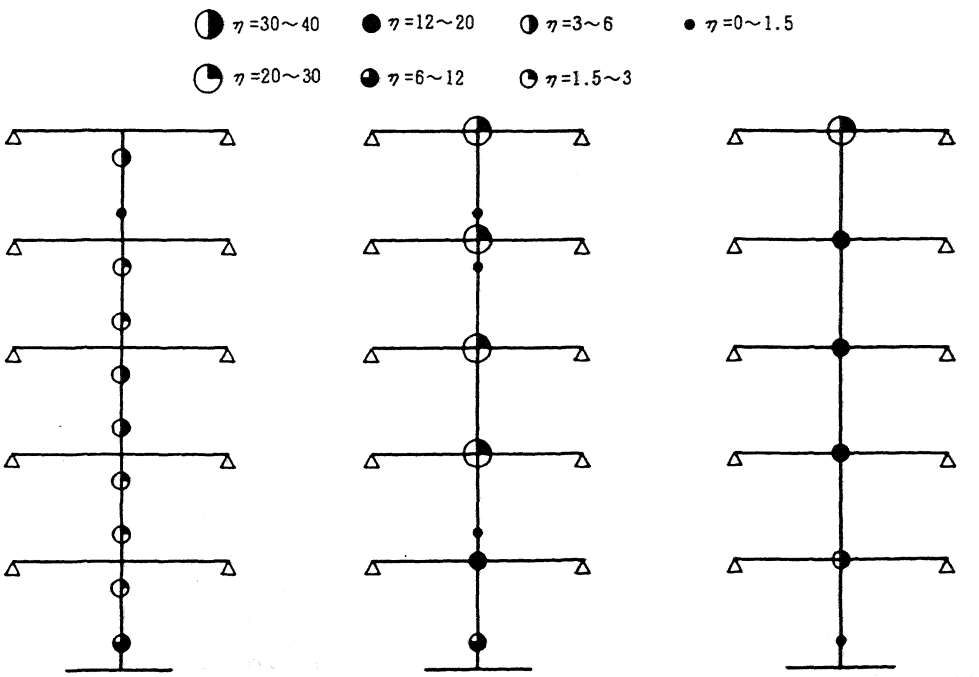
Looking at Fig.6, in case of Rpy>1, joint panels are almost within elastic state, and α1' is needed more than 0.46 against the El Centro record. On the other hand, as Rpy becomes smaller, α1' was reduced to 0.25 (Rpy=0.5), since the both joint panel and column absorb energy. However, in case of Rpy<0.5, it is necessary for α1' to be more than 0.4, since only joint panel absorb energy.

From the results of Analysis(C), it can be said that the most preferable Rpy under sever earthquakes is 0.5-0.6 to make a lower α1' of structure, since failure mechanism of the structural model having this Rpy (0.5-0.6) can make the optimum damage distribution to each member as shown in Fig.7 (2).

3.5 The seismic performance under medium earthquakes

In the Analyses (A),(B), and (C), the effect of weak beam-to-column joint on seismic performance were discussed under severe earthquakes. Here, there arises a significant question wether weak joint panel structures maintain or not the serviceability of a structure under medium earthquakes.

The Analysis (D) was carried out to examine the seismic performance of weak joint panel structures in



① Column Failure Mode (R_{py}=1.1, α_1'=0.46)

② Joint Panel and Column Failure Mode (R_{py}=0.5, α_1'=0.25)

③ Joint Panel Failure Mode (R_{py}=0.4, α_1'=0.41)

Fig. 7 Damage Distribution in terms of η

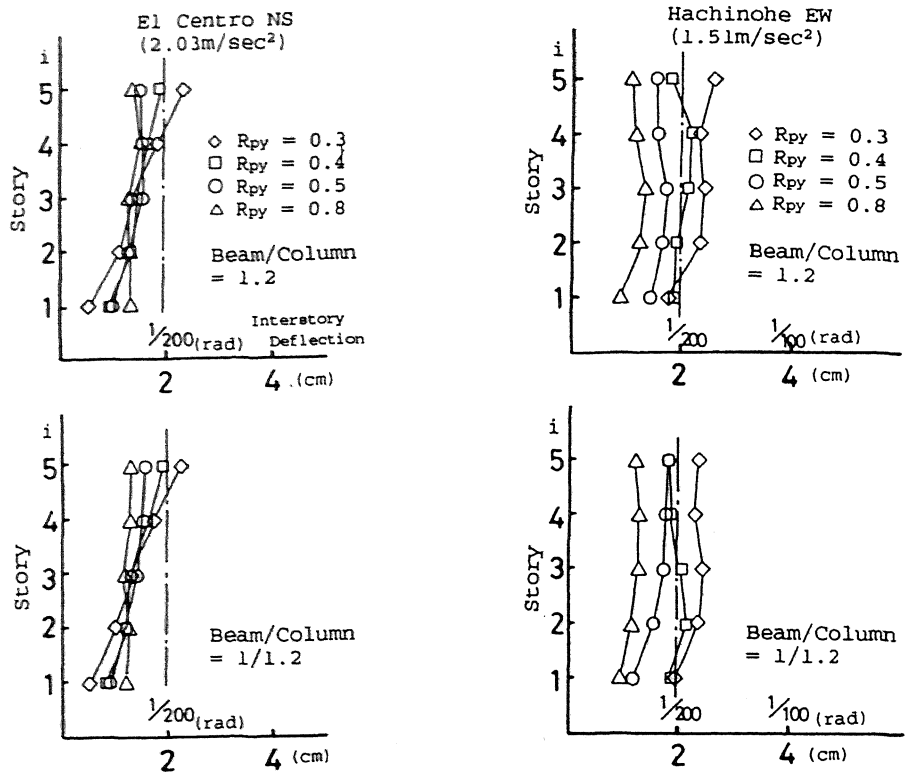


Fig. 8 Interstory Deflection against Medium Earthquakes

terms of the maximum interstory drift under medium earthquakes. In this model, major parameter Rpy were 0.3, 0.4, 0.5, and 0.8, and the strength ratios of beam to column were set to be 1.2 and 1/1.2. The used earthquake records were the El Centro and Hachinohe whose maximum velocities were scaled to 0.25 m/sec. to represent a medium level of earthquake ground motions.

Fig. 8 shows the relationships between Rpy and interstory deflection against medium earthquakes. From this figure, it would be concluded that Rpy should be larger than 0.5 if the upper limit of interstory drift angle is required to be 1/200 against medium earthquake as a serviceability limit.

4 CONCLUSIONS

In this paper, the effect of weak beam-to-column joint panels on seismic performance of steel building structures was investigated through the dynamic response analysis based on the energy concept. The following conclusions were obtained:

1) The total energy input represented by V_E depends primarily on the fundamental natural periods T of the systems, and barely on the predominant failure modes.

2) The weak joint panel structures were effective to preclude damage concentration into a particular story.

3) The preferable Rpy to make a lower $\alpha'1$-value of steel building structure was 0.5-0.6 under severe earthquakes.

4) Rpy should be larger than 0.5 against medium earthquake as a serviceability limit.

From the results of analyses considering energy input of the both severe and medium earthquakes, it can be concluded that the most preferable Rpy of steel building structure is 0.6.

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