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Seismic damage concentration in steel building structures with weak beam-to-column joint panels

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ABSTRACT

It is, in general, recommended that beam-to-column joint panels of moment-resisting steel building structures should remain elastic under seismic design forces representing medium earthquake effects. To keep this requirement, particularly in using H-shaped beams and columns, strengthening of beam-to-column joint panels with cover-plates or stiffeners is often needed. On the other hand, when no strengthening of beam-to-column joint panels is given, the seismic failure mode of this type of structures may be specified by the yielding of the weak joint panels prior to that of the adjacent beams or columns. Here, there arises a significant question whether weak joint panels are preferable or not under severe earthquake ground motions. This paper gives an answer to the above question through the dynamic time-history response analysis with an appropriate structural modeling, and by using a concept of seismic energy input.

INTRODUCTION

In a moment-resisting steel building structure, the strength of beam-to-column joint panels is a key factor that would determine the failure mode of the structure under severe earthquake ground motions. 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 forces representing the load effect of medium earthquakes. However, such strengthening as with cover plates or stiffeners usually requires difficult and tedious design work, and further the effectiveness of the strengthening is sometimes doubtful.

On the other hand, when no strengthening of joint panels is given, the panel yield strength is, probably in almost all cases, less than the larger yield strength of beams and columns. Thus, the seismic failure mode in those cases may be determined by the joint panel yielding prior to the yielding of the adjacent beams and columns; 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.

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At the present time, the "joint panel failure mode" is not yet recognized to be recommendable since the effect of joint panel strength on the overall seismic behavior has not yet fully understood. Recently a few studies have been carried out regarding the effect of strength of joint panels on the strength and ductility of steel frames (Nakao 1990; Tanaka 1990). In these studies, ratios of panel yield strength to the lesser yield strength of adjacent beams and columns are presented as a recommendable lower limit. However, the ratios should be determined through a parametric study on the effect of panel strength on the characteristics of seismic damage concentration in structures.

The primary object of this paper is then to investigate the influence of strength of beam-to-column joint panels on seismic behavior of steel structures and to propose a minimum strength of joint panels, which is required to keep the structures sound under both medium and severe earthquake conditions.

ANALYSIS

Analytical models

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(a). This model is assumed to be extracted from a horizontally infinite and equally spanned moment-resisting plane frame as shown in Fig. 1(b). Further, it is assumed that each particle of mass has the quantity of m. As characteristics of the members used in the model, only flexural deformation is considered for beams and columns, and only shear deformation for joint panels.

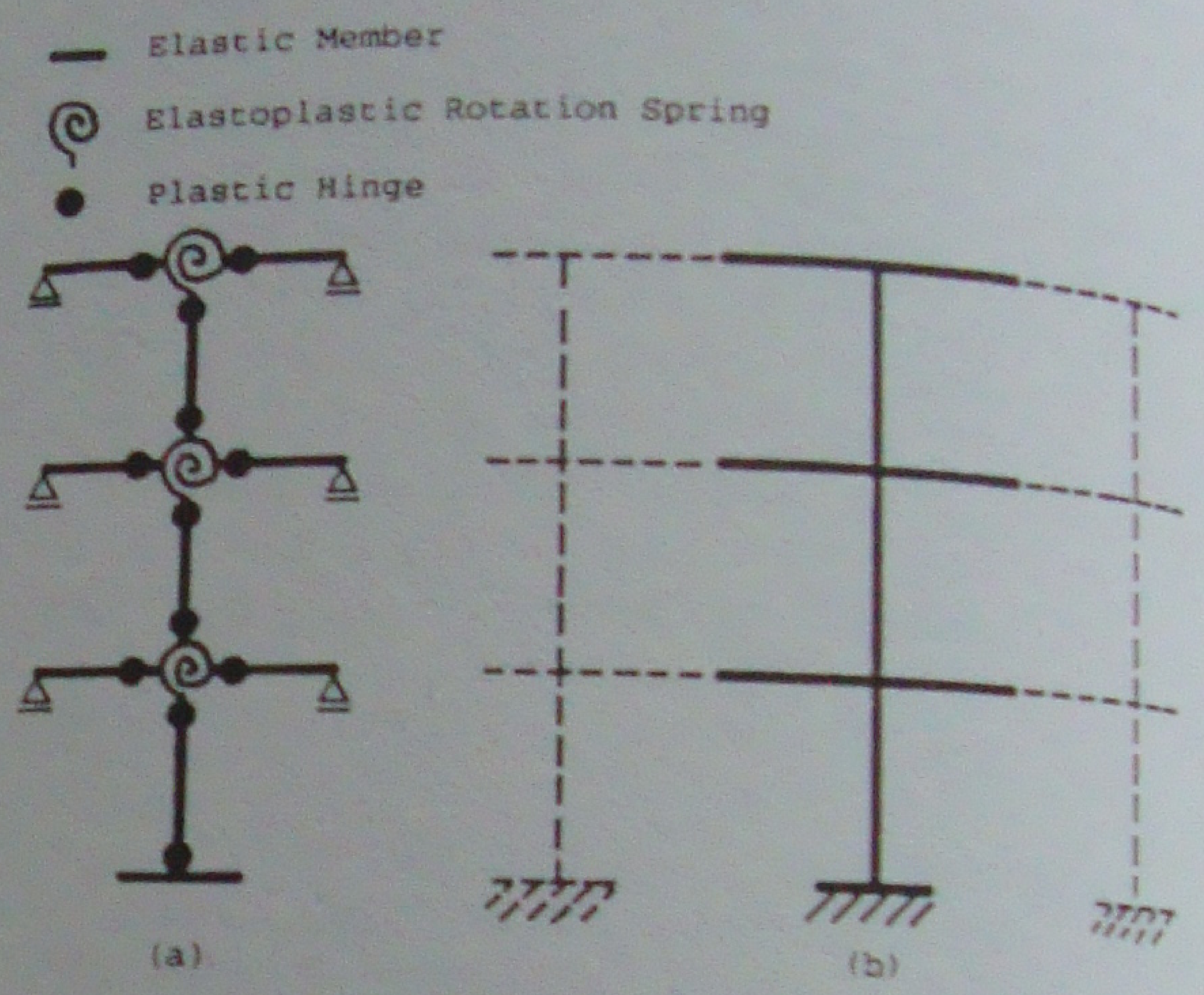

Fig. 1 Vibration Model for Analyses

In addition, axial forces in the columns are neglected. As 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 springs representing shear deformation of the panels are considered. The restoring force characteristics of these inelastic elements are assumed to have elastoplastic bi-linear relationships.

Three groups of analysis

The analysis of this study is divided into three groups; Analyses A, B and C. The purposes and major parameters of each analysis are briefly summarized in Table 1.

The Analysis A was carried out to investigate the effect of failure mechanisms on total input energy into the model. Three different failure modes, that is, the 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.

The purpose of the Analysis B was 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.

Table 1 Purposes and Parameters for Analyses

	Analysis(A)	Analysis(B)	Analysis(C)
Story N	5,10	5	5
Base shear coefficient; α_1	0.1 0.2 0.3	0.2	0.25
Rpy Beam/column strength ratio	--- ---	0.3 - 1.3 1/1.4 - 1.4	0.3 - 1.2 1/1.2 1.2
Fundamental natural period	1.0(s)	1.0(s)	0.57 - 0.77(s)
Second slop of bi-linear	column,beam; 0% panel; 0%	column,beam; 0% panel; 3%	column,beam; 0.5% panel; 2%
Input earthquake ground motion unit(m/sec^2)	Hachinohe EW (Max1.83)	Hachinohe EW (Max1.83)	Hachinohe EW 0.5m/sec(Max.3.02) 0.25m/sec(Max.1.54) El Centro NS 0.5m/sec(Max.4.06) 0.25m/sec(Max.2.03)
Purpose of analysis	Effect of failure mechanism on input energy	Relation between Rpy and damage concentration	Seismic performance in terms of η and interstory drift

Here, the panel strength was represented by a new important parameter, Rpy, which is defined as follows (also see Fig.2);

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

where min[A,B] means the lesser of A and B, and where $_{Lb}M_{yi}$ and $_{Rb}M_{yi}$ are yield moments of left and right hand side beams adjacent to the joint panel, and $_{Uc}M_{yi}$ and $_{Lc}M_{yi+1}$ are yield moment of the upper and lower columns adjacent to the joint panel.

In this analysis, the values of Rpy were varied from 0.3 to 1.3. Also in this analysis, 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 the Analysis C, the model structures in which the joint panel yielding was ahead of the yielding of beams and columns were dealt with to obtain the seismic performance in terms of the maximum interstory drift and accumulated plastic deformations of the inelastic hinge elements. Further, to investigate the seismic performance under both medium and severe earthquakes, the maximum input velocities of 0.25m/sec and 0.5m/sec were chosen. As input waves, the two recorded ground motions, Hachinohe Earthquake 1968(EW) and El Centro Earthquake 1940(NS) were linearly scaled to have the two maximum velocities.

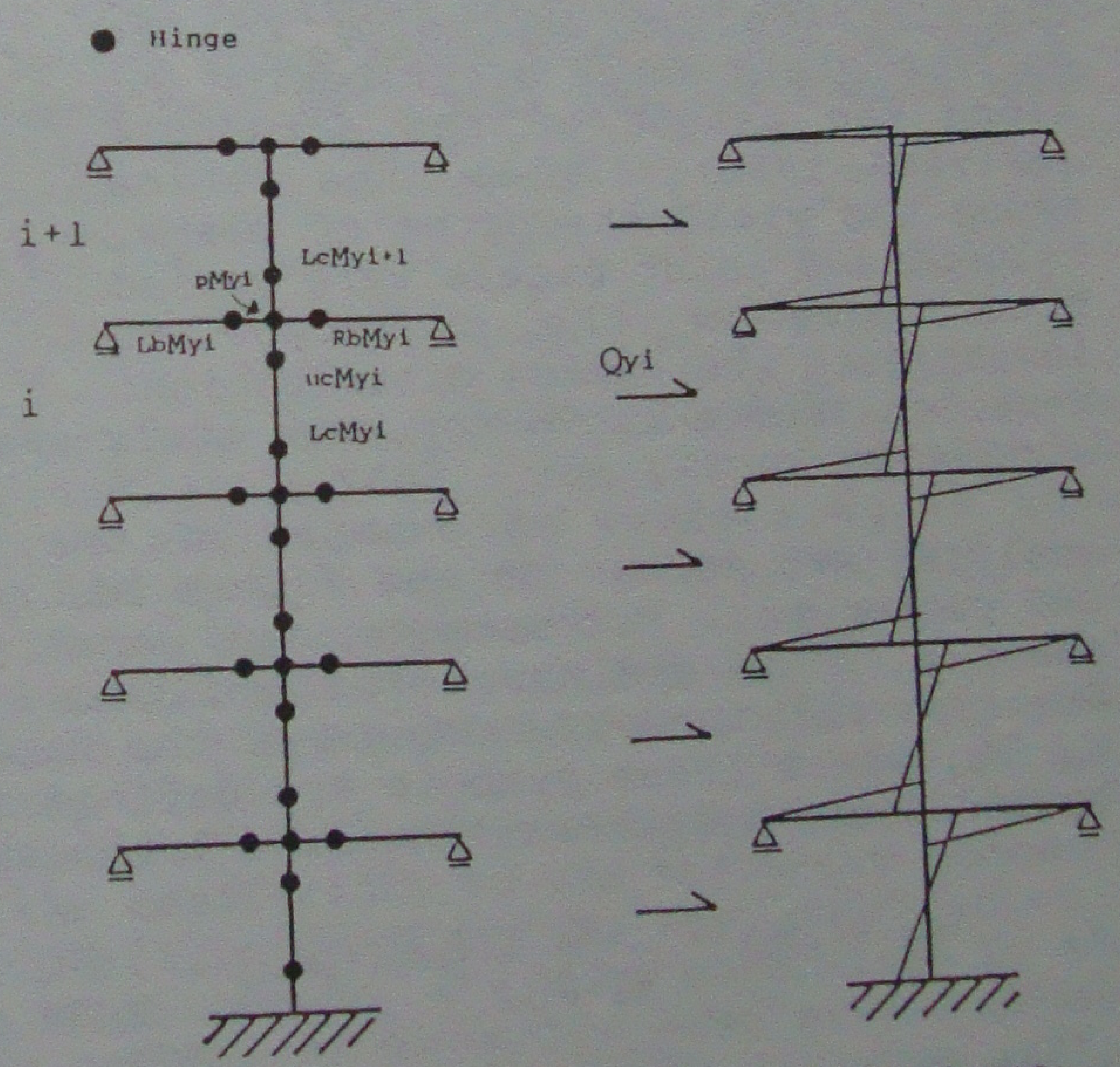

Fig. 2 Prototype Model Structure

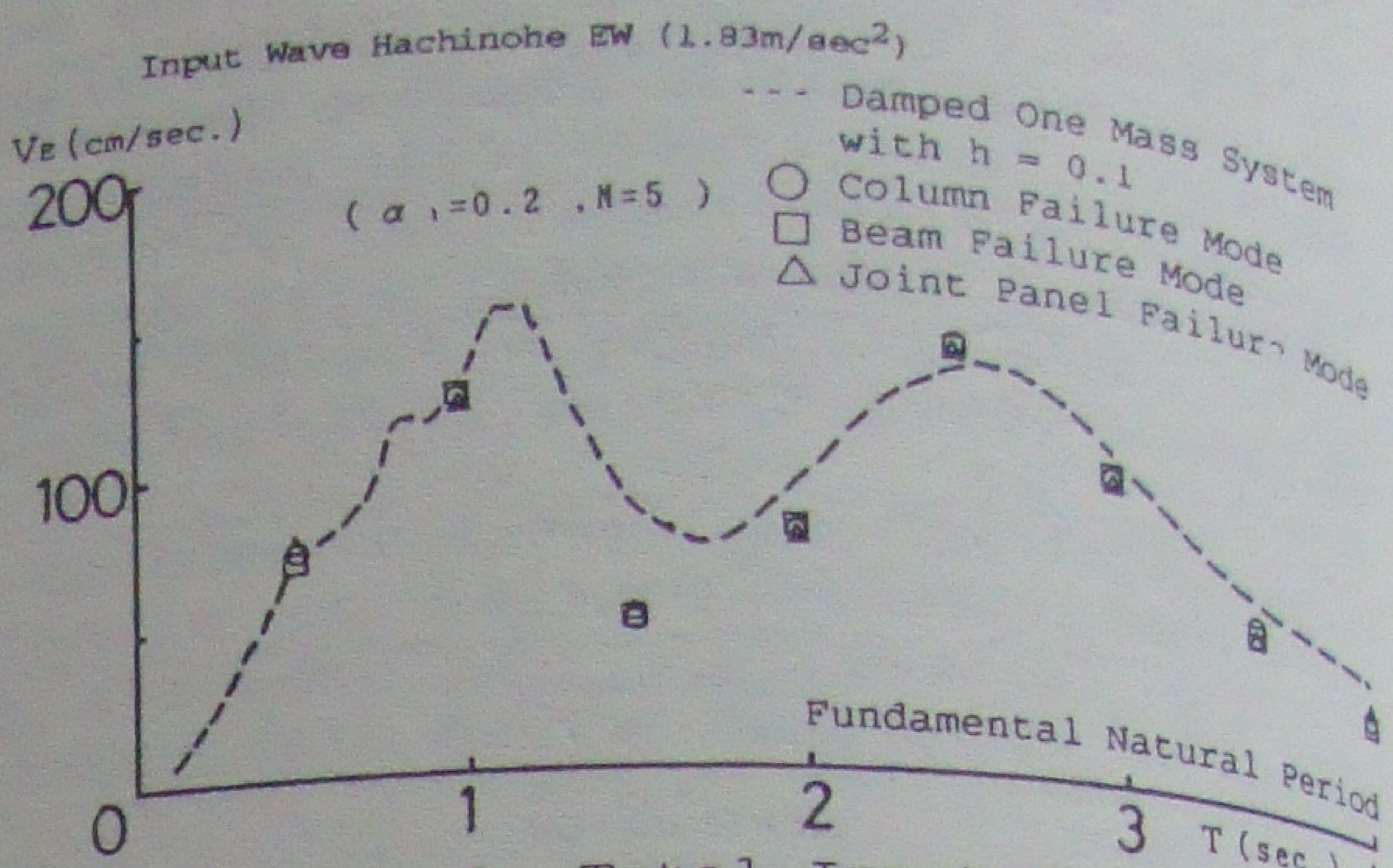

Fig. 3 Total Input Energy

Total input energy

Fig. 3 shows the most significant result of the Analysis A; this figure deals with the model structures having the base shear coefficient $\alpha_1 = 0.2$ and the number of stories $N = 5$. Here, the total input energy into the structures was normalized as the form of velocity V_E which is defined as

$$V_E = \sqrt{\frac{2E}{M}} \tag{2}$$

where E is the total input energy and M is the total mass of the system. From this figure, it can be seen that the total input energy represented by V_E depends on only the fundamental natural period T of the system and does not on the predominant failure modes. Also in this figure, the dashed line designates the VE 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.

Damage concentration

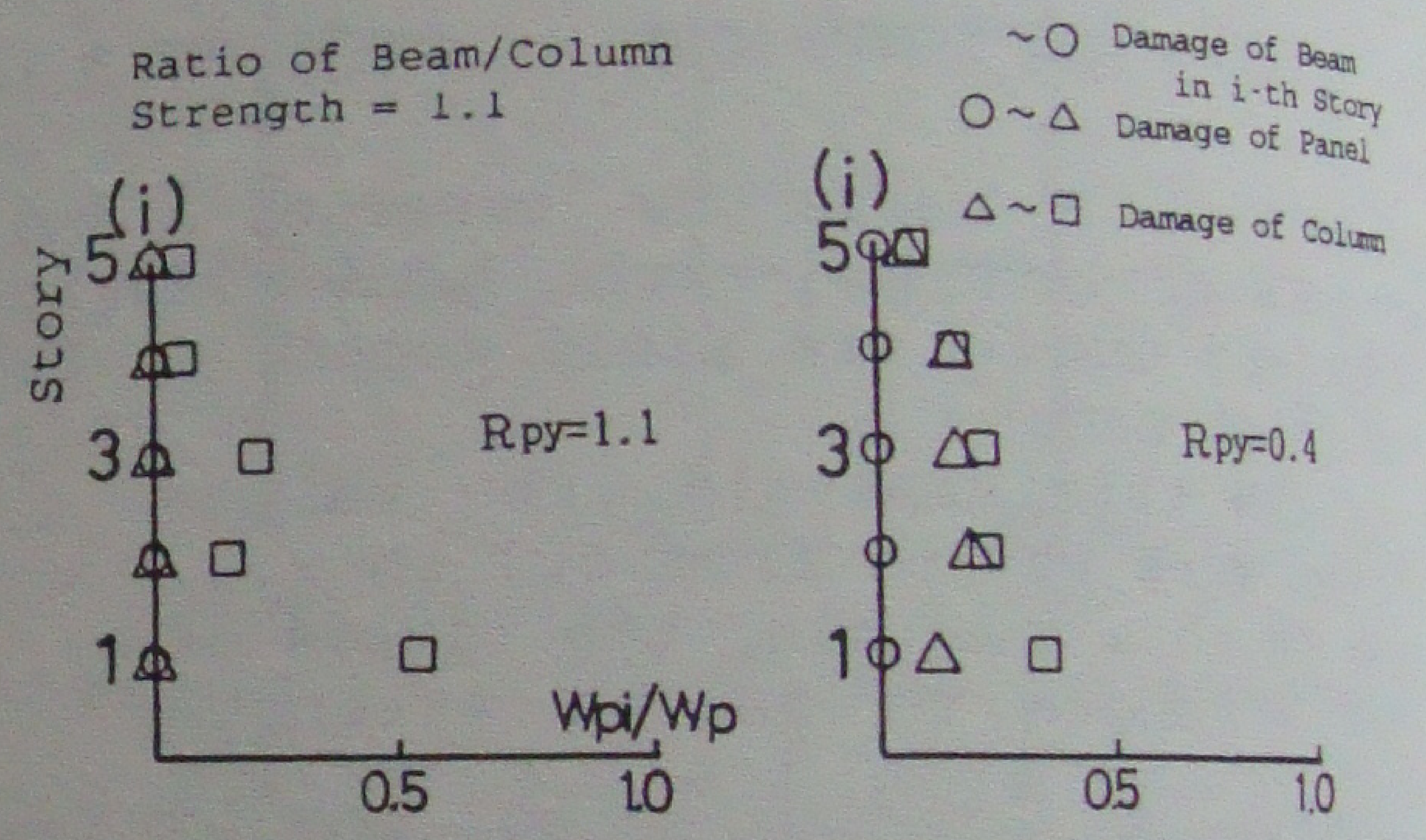

Fig. 4 Damage Distribution along the Height of Model Structures

Before discussing the results of the Analyses B and C, we should describe the structural model in more detail. Fig. 2 shows the prototype model structure in which the yield moments of beams and columns determined by the moment distribution corresponding to the optimum yield-shear distribution Qyi along the height of the structure (Akiyama 1985). Then, the ratios among the yield strengths of beams, columns and joint panels were varied from those of the prototype to investigate the damage distribution. Here, the above ratio for each story was set to be identical for one analyzed structure.

Fig. 4 shows an example of the results of the Analysis B. The two analyzed structures of the figure had 1.1 times stronger beam than those of the prototype. Furthermore, the panel yield ratios, Rpy, were set to be 1.1 and 0.4. 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

$$W_{pi} = {}_cW_{pi} + \frac{K_i}{K_i + K_{i+1}}\left({}_pW_{pi} + {}_bW_{pi}\right) + \frac{K_i}{K_i + K_{i-1}}\left({}_pW_{pi-1} + {}_bW_{pi-1}\right) \tag{3}$$

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, ${}_{Uc}W_{pi} + {}_{Lc}W_{pi}$. Furthermore, the total input plastic energy into a structure, W_p, is given by

$$W_p = \sum_{i=1}^{N} W_{pi} \tag{4}$$

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.

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);

$$\frac{W_{pk}}{W_p} = \frac{S_k \cdot P_k^n}{\sum_{j=1}^{N} S_j \cdot P_j^n} \tag{5}$$

where

n = damage concentration factor, $P_j = (\alpha_j/\alpha_1)/\bar{\alpha}_j$,

$\bar{\alpha}_j$ = optimum yield-shear force coefficient distribution, and

$$S_j = \left(\sum_{k=j}^{N} m_k/M\right)^2 \bar{\alpha}_j^2 \,(K_1/K_j).$$

The damage concentration factor, n, is then obtained from Eq. 5 by using the results of response analysis where only the story shear coefficient of the k-th story, α_k, is changed from the optimum one, $\bar{\alpha}_k$. That is, the damage concentration factor, n, is expressed as

$$n = -\ln\left\{\frac{b(1-a)}{a(1-b)}\right\} / \ln P_d \tag{6}$$

where a is given by W_{pk}/W_p with the optimum shear coefficient $\bar{\alpha}_k$ and b is W_{pk}/W_p with the changed shear coefficient, $\alpha_k = P_d\bar{\alpha}_k$. 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 1985).

Now, the factor n was evaluated on the structure models having different Rpy-values 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 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.

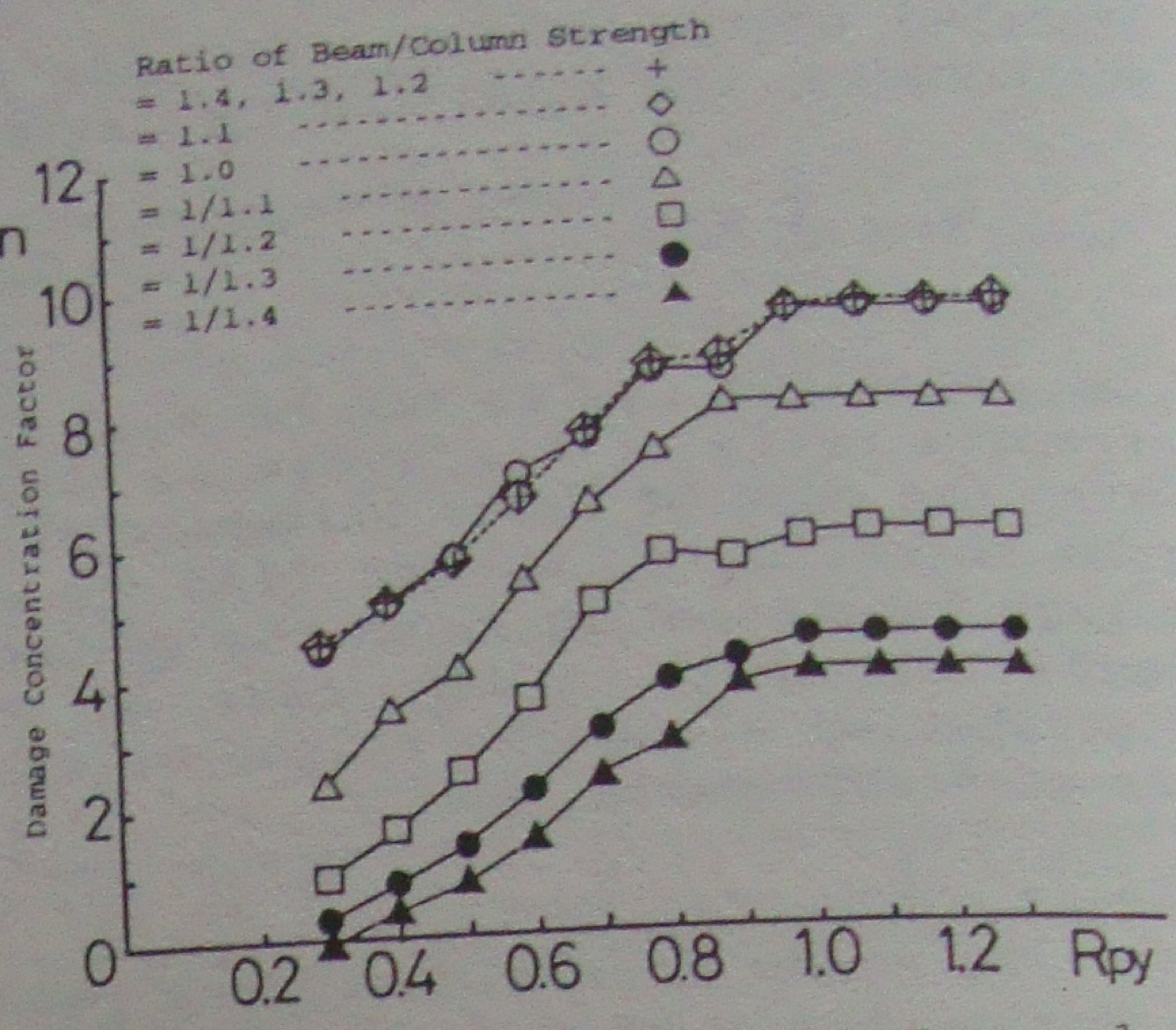

Fig. 5 Relationship between Rpy and n

Seismic performance

The analysis C was carried out to examine the seismic performance of structures having weak joint panels in terms of plastic deformations of hinge- and spring-elements and interstory drifts. Here, to indicate the amount of plastic deformations of hinge and spring-elements, we used the " cumulative inelastic deformation ratio, η " which is defined as follows;

$$\eta = \frac{{}_jW_{pi}}{{}_jM_{yi} \cdot {}_j\theta_{yi}} \qquad (7)$$

where j expresses each member of the column, beam and joint panel, and where jMyi and jθyi is the yield moment and yield rotational angle of the i-th story's j member.

Now, Figs. 6 demonstrates some examples of the analytical results in terms of obtained η regarding inelastic elements in beams, columns and joint panels; this figure shows just the results obtained against the input of El Centro record having the scaled maximum velocity of 0.5m/sec. In this analysis, the panel yield ratios, Rpy, were set to be 1.2, 1.0, 0.7 and 0.3, and the strength ratio of beams to columns was chosen to be 1.2.

Looking at Fig. 6, we can say that η of the panels becomes larger as Rpy becomes smaller, and corresponding to this inclination, η of beams and columns decreases. Furthermore, it can be found that in the case of Rpy >1.0, the values of η of the columns become larger than those in the case where Rpy<1.0. These values exceed the value of η=12 which is generally accepted as the upper limit of H-shaped columns (S.C. 1990).

On the other hand, when Rpy is 0.7, the values of η of the columns do not reach the upper limit except for the bottom of the first story column, since the weak panels absorb a considerable amount of energy. Therefore, it can be said that weak panels is effective to reduce the damage of beams and columns, in particular to prevent the failure of columns.

Fig. 7 shows the relationships between Rpy and the average of η of all panels in each model. In addition, recent data obtained from monotonic loading tests on cruciform beam-to-column joint specimens (Matsumoto 1990) are

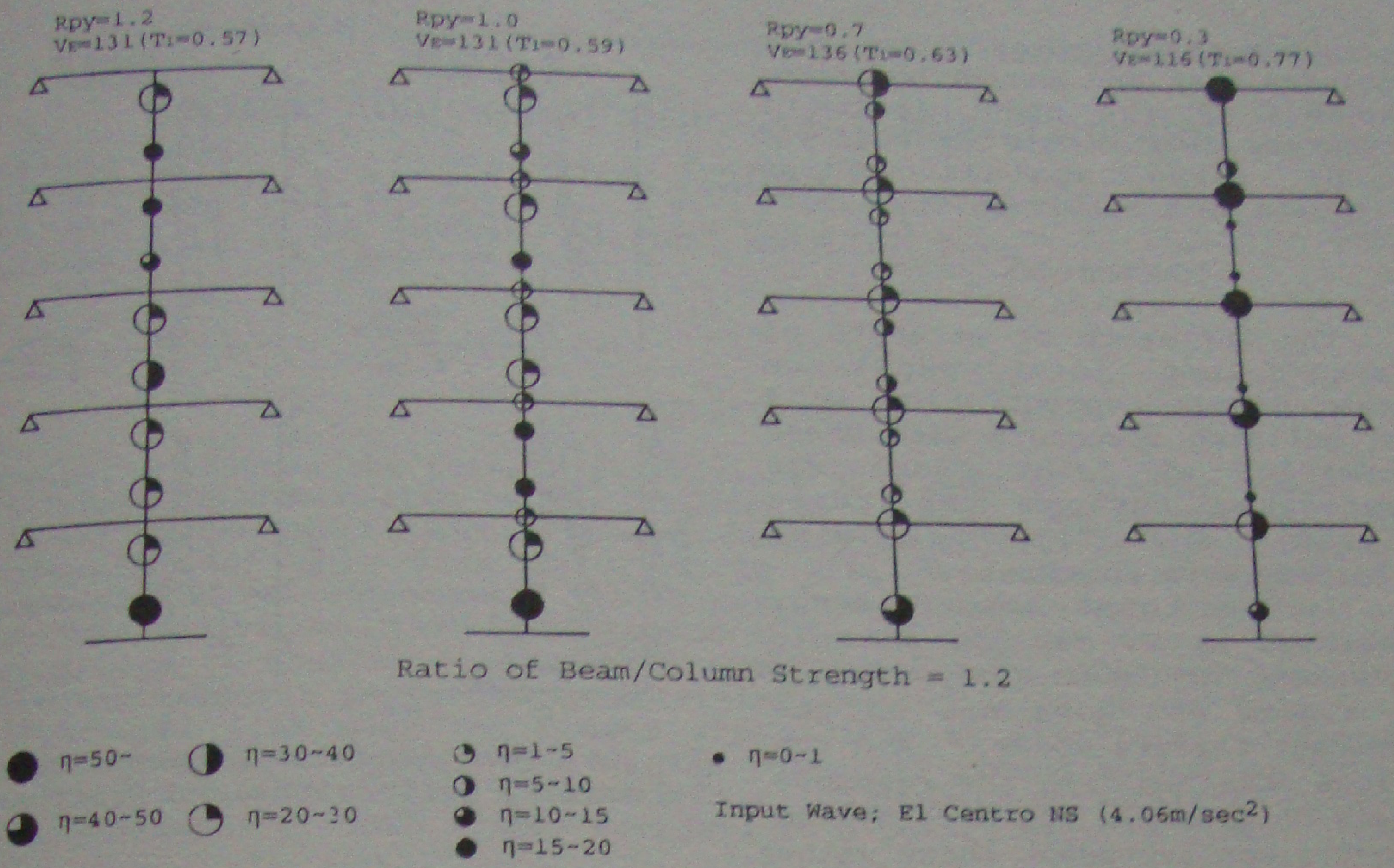

Fig. 6 η of Each Member against Severe Earthquake

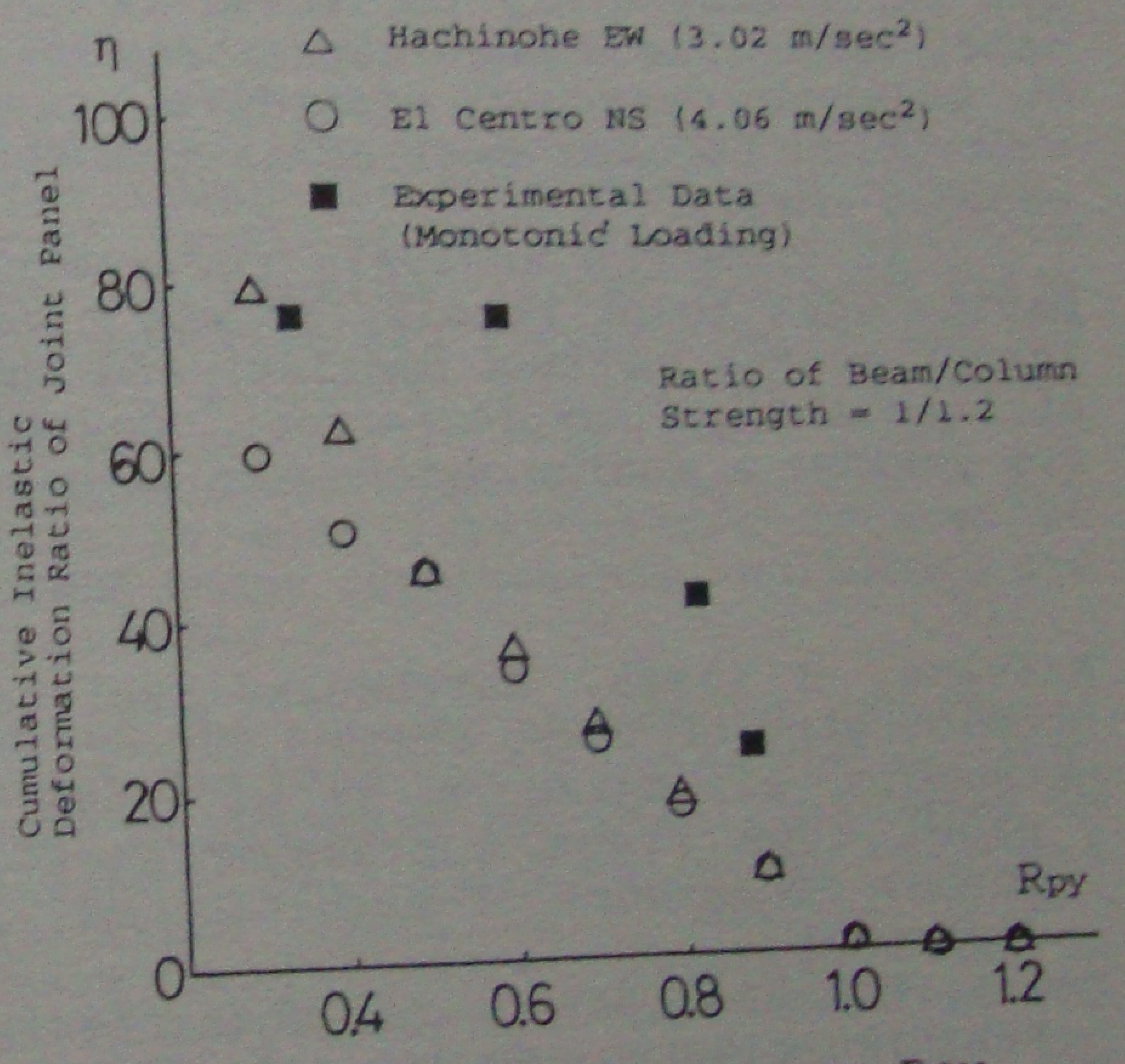

Fig. 7 Relationships between Rpy and η of Joint Panels

also plotted in this figure. By these experimental data, the upper limit of η to lose load-carrying capacity is around 75, whereas the analytical η reaches about 80 at Rpy of 0.3. Thus, by this comparison, the joint panel with Rpy=0.3 seems to be critical against such severe earthquake as the scaled Hachinohe. However, the experimental η of 75 was attained under monotonic loading so that attainable η under reversed and alternative cyclic loading simulating seismic responses would be larger than η=75, because the monotonic deformation capacity of panels can be demonstrated in both the plus and minus loading directions. By this consideration, even when the panel yield ratio, Rpy, is 0.3, a dangerous failure of the panel would not occur in the structure having the base shear coefficient of 0.25.

As the other result of the Analysis C, Fig. 8 shows the relationships between Rpy and interstory deflection against medium earthquakes. In this part of the analysis, the panel yield ratio, Rpy, were 0.3, 0.4, 0.5, and 0.8. 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.

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 serviceablity limit.

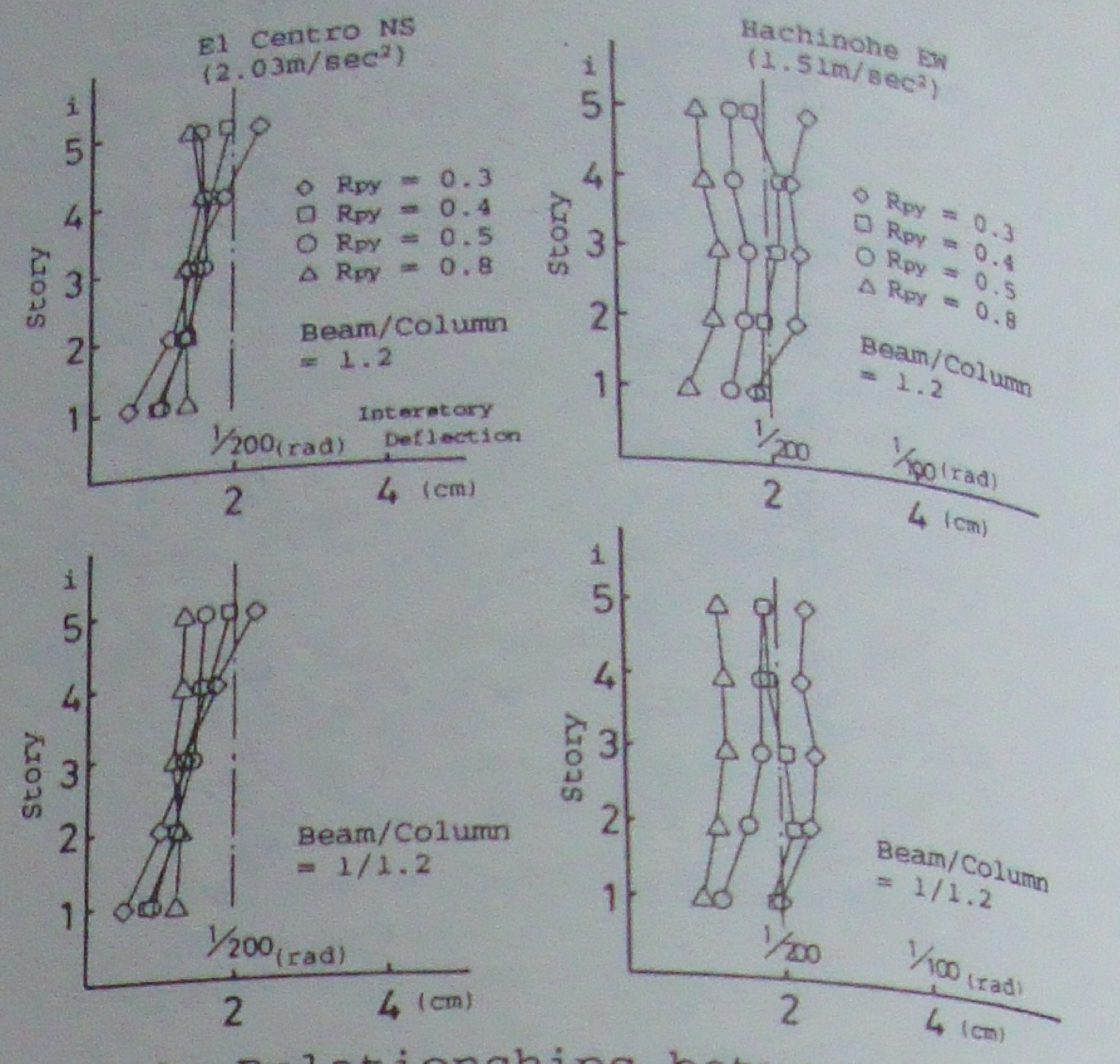

Fig. 8 Relationships between Rpy and Interstory Deflections and Drift Angles

CONCLUSIONS

The influence of strength of beam-to-column joint panel on seismic damage concentration in a steel building structure with shear deformation of joint panel was investigated through the three analyses, and the following conclusions were obtained.

1) The total input energy represented by VE depends on only the fundamental natural period T of the system and does not on the predominant failure modes. And the VE values obtained from the response analysis on a damped one mass system with h=0.1 can represent these values.

2) Regarding the relation between Rpy and seismic damage concentration, in the region of Rpy<1.0, there is a tendency for damage concentration into the particular 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. The damage concentration becomes smaller as the ratio of beam strength to column strength becomes smaller.

3) From the analysis to examine the seismic performance of structure having weak joint panels, it was cleared that η of the joint panels becomes larger as Rpy becomes smaller, and that corresponding to this inclination, η of beam and column decreases. On the other hand, Rpy should be larger than 0.5 against a medium earthquake as a serviceablity limit.

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