INFLUENCE OF THE ELASTO-PLASTIC BEHAVIOR OF COLUMN BASES ON THE ULTIMATE EARTHQUAKE RESISTANCE OF MULTI-STORY STEEL MOMENT FRAMES

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SUMMARY

Column base is one of the most important elements of steel moment resisting frames. It is important to investigate the influence of elasto-plastic behavior of column bases on the ultimate earthquake resistance of steel moment resisting frames. In this paper, a series of inelastic response analysis of multi-story steel moment resisting frames with exposed type column bases, especially weak anchor bolt type column bases, were carried out.

Analytical models were weak beam type 9-story moment resisting frames and weak panel type 9 story moment resisting frames, and the deformation capacities of members were the parameters. Column bases of the analytical models were set as exposed type column base, fixed type bases and pin type base. The hysteresis characteristics of members and column bases applied in the analyses were assumed as realistic as possible on the basis of experimental results.

In the analysis, responses of the analytical models were calculated for NS component of the 1940 El Centro record and EW component of the 1968 Hachinohe record. The main parameter of the analyses was the amplification factor, which was multiplied on the accelerograms. Thus, the continuative analytical results were derived, that corresponds to the energy input revel.

From the analytical results, following conclusions were obtained. 1) The intensity of the damage concentration at the structural element of the 1st story in the frames with exposed type column bases were much less than that of the frames with pin type column bases and the frames with fixed column bases. 2) Ultimate earthquake resistance of the frames with exposed type column bases is similar to that of the frames with pin type column bases. 3) Ultimate earthquake resistance of the frames with exposed type column bases is much better than that of the frames with fixed column bases. 4) Absorbed energy by the exposed type column bases was much less than other structural elements.

INTRODUCTION

Recently plastic design method is introduced in seismic design of steel moment resisting frames. Column base is one of the most important elements of steel moment resisting frames. It is important to investigate the influence of elasto-plastic behavior of column bases on the ultimate earthquake resistance of steel moment resisting frames. In this paper, a series of inelastic response analysis of multi-story steel moment resisting frames with exposed type column bases, especially weak anchor bolt type column bases, were carried out. The hysteresis models of members used in this study are based on the experimental results. So analytical results are reflected the realistic behaviors of steel members, i.e. Bauschinger effect, local buckling etc. Thus, the ultimate earthquake resistances of steel moment resisting frames are appropriately evaluated.

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ANALYTICAL MODEL

Analytical models, shown in Fig.1, are 9-story infinitely continued plane frames. In the models, columns have rectangular hollow section, beams have wide-flange section, and panel-zones have same section as that of the lower columns of under story. Analytical models are designed by the ultimate strength design method based on the individual D_S-value of each member. The D_S-Value is the reducing factor of required restoring force in plastic design. It is determined by the plastic deformation capacity. Eq (1) presents the basic relationship between D_S-value and required restoring force.

$$_pF_{d,i} = D_S \cdot {_eF_{d,i}} \tag{1}$$

where $_pF_{d,i}$: the required restoring force of i-th story in the plastic design.

$_eF_{d,i}$: the required restoring force of i-th story in the elastic design.

Now, it is assumed that, frames plastify with the plastification of only one kind of element such as columns, beams or panel zone, and other elements stay in elastic zone. The D_S-value of the frame depends only on the assumed plastified element. Then, applying that assumption on each element, the individual D_S-value can be given. So, the individual D_S-value means the reducing factor of required restoring force of members based on the plastic deformation capacity of individual members. Current Monograph of AIJ (Architectural Institute of Japan) indicates the individual D_S-value corresponding to three structural ranks, such as structural rank 1, 2, and 3 according to the required plastic deformation capacity of members. [AIJ 1990]

Parameters of analytical models are the type of the column base and individual D_S-value of members. The type of column base is exposed type column base, fixed type bases and pin type base. The individual D_S-value of members are chosen as follows;.

1)Individual D_S-values of columns,

$$D_S = 0.30, 0.42.$$

These values correspond to the structural rank 1 and 2. Required plastic deformation capacities of columns are 6 and 3. Definition of the required plastic deformation capacity η_0 is shown in Fig.3.

2) Individual D_S-values of beams,

$$D_S = 0.25, 0.37.$$

These values correspond to the structural rank 1 and 3. Required plastic deformation capacities of beams are 3 and 0.

Irrespective of structural rank, individual D_S-value of panel zone is 0.30. Individual D_S-value of exposed type column bases is set to the same value of beams of the 1st story, and elastic stiffness of expose type column bases is set to the same value of columns of the 1st story.

Analytical models are named as follows.
$\underline{AX_1}\underline{X_2}\underline{X_3}$
X_1 indicates the structural rank of columns.
X_2 indicates the structural rank of beams.
X_3 indicates the type of column base. E means exposed type, F means fixed type and P means pin type.
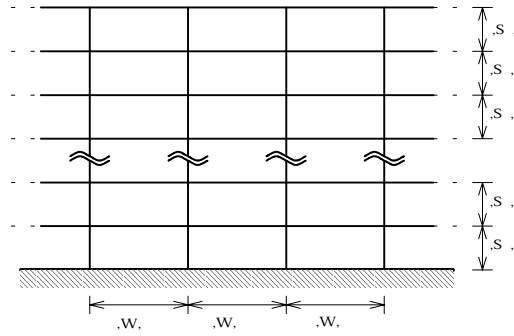List of the analytical models is shown in Table 1.
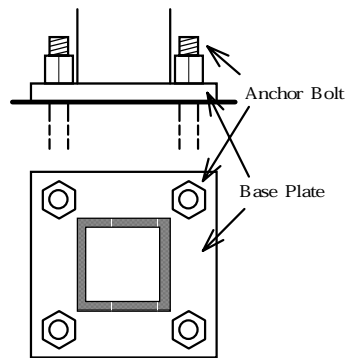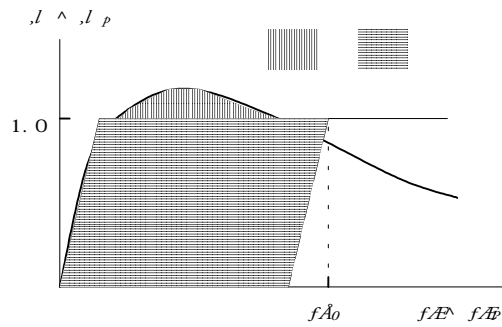
Fig.1 Analytical Model

Anchor Bolt

Base Plate

Fig.2 Shape of the Exposed Type Column Base

Fig.3 Definition of η_0

Table 1 List of the Analytical Models

| | Structural Rank | | | Section of Members Column and | |
	Column	Beam	Story	Panel Zone	Beam
A11B			7, 8, 9	□-53.4x1.78	H-64.1x22.4x1.07x1.60
A11F	,	P	4, 5, 6	□-61.5x2.12	H-73.8x25.9x1.23x1.85
A11P			1, 2, 3	□-65.2x2.33	H-78.2x27.3x1.30x1.95
A13B			7, 8, 9	□-53.4x1.78	H-64.1x40.0x1.03x1.54
A13F	,	P	4, 5, 6	□-61.5x2.12	H-73.8x46.0x1.18x1.77
A13P			1, 2, 3	□-65.2x2.33	H-78.2x54.1x1.39x2.08
A21B			7, 8, 9	□-57.8x1.52	H-69.3x19.0x1.05x1.58
A21F	,	Q	4, 5, 6	□-65.9x1.83	H-79.1x22.3x1.24x1.86
A21P			1, 2, 3	□-70.0x2.00	H-84.0x23.8x1.32x1.98
A23B			7, 8, 9	□-57.8x1.52	H-69.3x35.8x0.99x1.49
A23F	,	Q	4, 5, 6	□-65.9x1.83	H-79.1x42.0x1.17x1.75

HYSTERESIS CHARACTERISTICS OF MEMBERS

The hysteresis characteristic of steel members under cyclic loading consists of skeleton part, Bauschinger part, and elastically unloading part, as shown in Fig.4. The skeleton part is equivalent to the load-deflection relationship under monotonic loading, [Kato et.al. 1968]. The load-deflection relationship of steel members under monotonic loading is able to be analyzed including the deteriorating behavior governed by local buckling, [Yamada et.al. 1995]. And the hysteresis characteristic of the Bauschinger part is simply modeled by Akiyama and Takahashi [Akiyama et.al. 1990]. Thus, load-deflection relationships of members are modeled tetra-linear as shown in Fig.5, and applied as the skeleton part of the hysteresis characteristics. Examples of the calculated load-deflection relationship of members are shown in Fig.6. The Akiyama and Takahashi's model are applied as the Bauschinger part of the hysteresis characteristics. On the other hand, the hysteresis characteristic of exposed column base is modeled a type of slip model as shown in Fig.7. This model includes the effect of the friction caused by the axial force and the effect of strain hardening of the anchor bolts.

ANALYTICAL METHOD

In the analysis, responses of the analytical models are calculated for NS component of the 1940 El Centro(U.S.) record and EW component of the 1968 Hachinohe(Japan) record. The main parameter of the analyses is the amplification factor, which is multiplied to the accelerograms. Thus, the analytical results are continuously obtained, according to the energy input level. The collapse of frame is defined as the state that any one of the elements loses its restoring force. And, at the end of analysis when frame reaches just before collapse, the state of the frame defined as the ultimate state.

ANALYTICAL RESULTS

Damage distribution

Examples of the damage distributions of the analytical models at the ultimate state are shown in Fig.8. The vertical axis shows the position of the story, and the horizontal axis shows the percentage of the absorbed energy by each structural element in the story to the total absorbed energy of frames. In this analysis, analytical models formed full yield mechanisms. Thus A11 models and A21 models formed weak beam type mechanism and A13 models and A23 models formed weak panel type mechanism.

In analytical models with exposed type column bases, damage disperses over beams and panels. And absorbed energy by the exposed type column bases was much less than other structural elements. On the other hand, in analytical models with fixed type column bases, damage concentrates the lower end of column of the 1st story. In analytical models with pin type column bases, damage concentrates the upper beam or panel zone of the 1st story. So it is clarified that, the intensity of the damage concentration at the structural element of the 1st story in the frames with exposed type column bases were much less than that of the frames with pin type column bases and the frames with fixed column bases.

Ultimate earthquake resistance

Ultimate earthquake resistance of analytical models is evaluated in terms of the equivalent velocity of the total energy input at ultimate state V_{max} to the equivalent velocity of the required absorbing energy V_{EM}. The equivalent velocity V_E is defined by Eq.(2). [Akiyama (1985)]

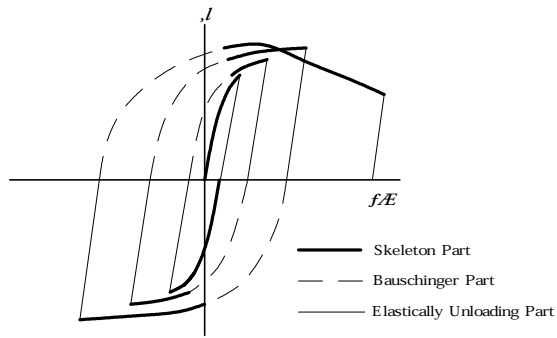$$V_E = \sqrt{2E/M} \tag{2}$$

where E :energy.

   M :mass.

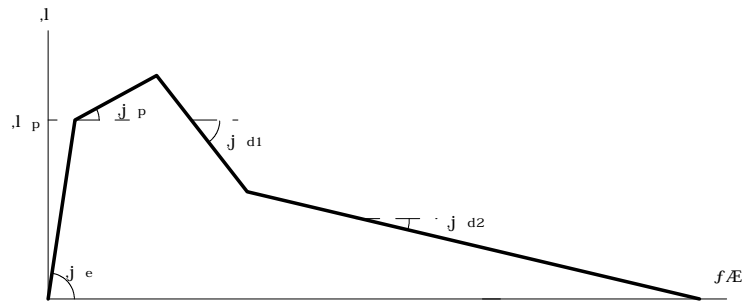**Fig.4 Typical Hysteresis Characteristics of Members**

- Skeleton Part
- Bauschinger Part
- Elastically Unloading Part

Fig.5 Modeled Skeleton Curve of Columns and Beams

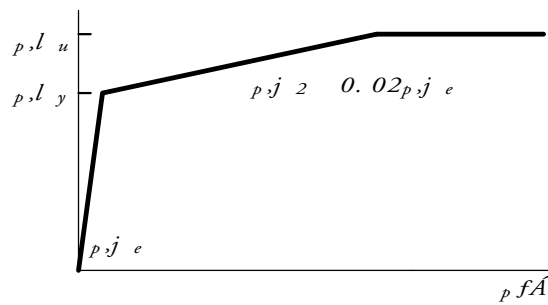$$_p\,j_2 \quad 0.\,02_p\,j_e$$

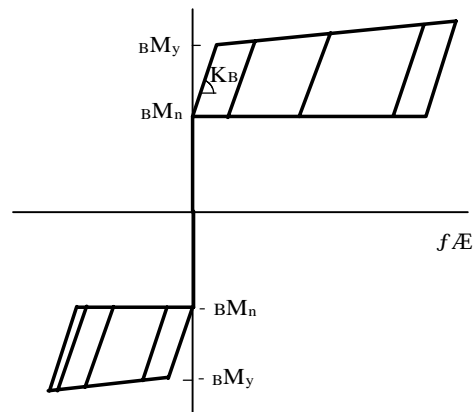**Fig.6 Modeled Skeleton Curve of Panel Zones**

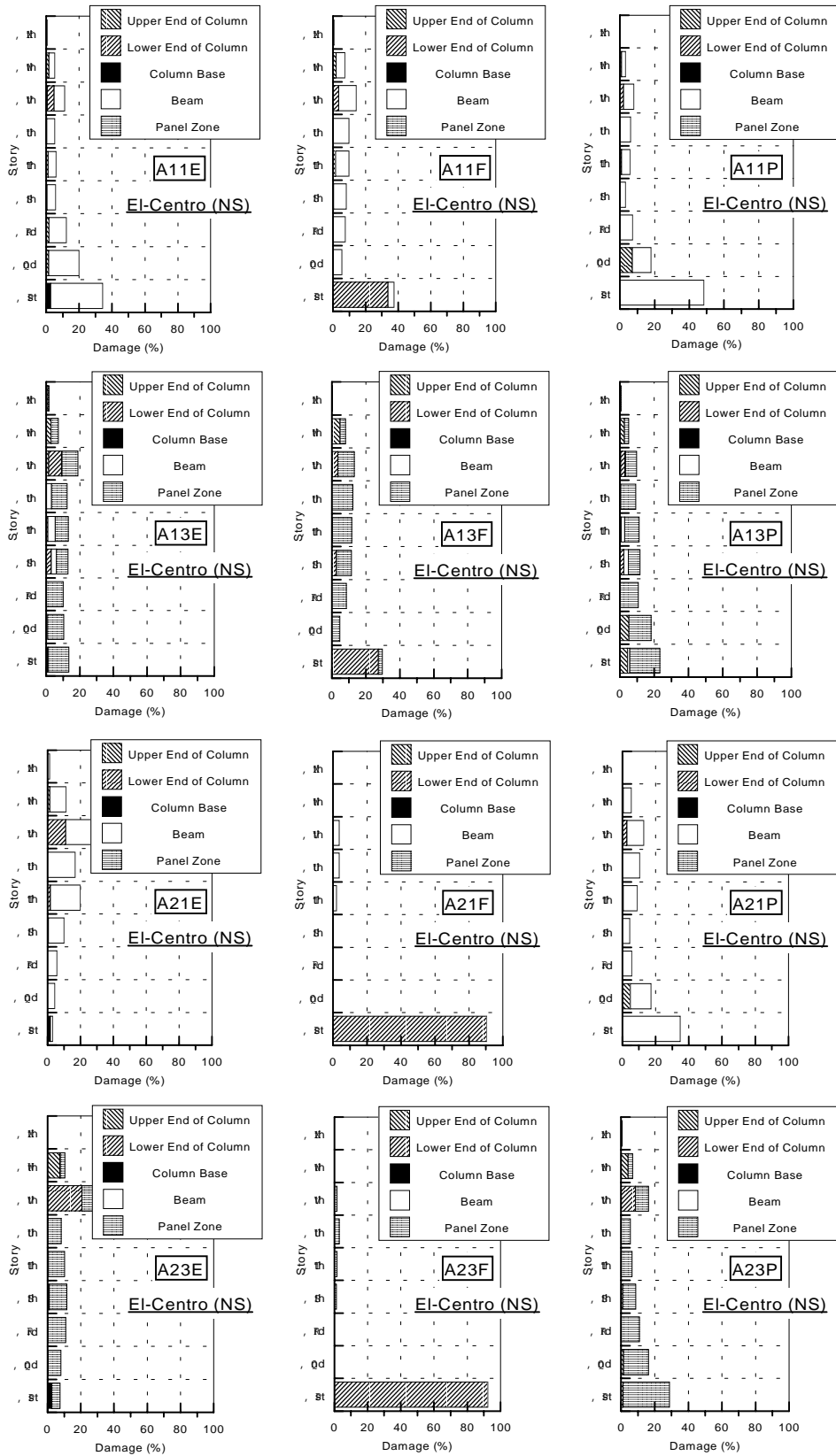**Fig.7 Hysteresis Characteristics of Exposed Type Column Bases**

Fig.8 Examples of the Damage Distributions (Ultimate State)

V_{max}/V_{EM} are plotted in Fig.9 in comparison of the type of column base. The ultimate earthquake resistances of analytical models with exposed type column bases greatly exceed to the required absorbing energy. But the ultimate earthquake resistance of analytical models with fixed column bases and columns of structural rank 2 are

slightly less than the required absorbing energy. So it can be conclude that, the ultimate earthquake resistance of the frames with exposed type column bases is much better than that of the frames with fixed column bases. On the other hand, the ultimate earthquake resistances of analytical models with pin type column bases are slightly less than that of analytical models with exposed type column bases and greatly exceed to the required absorbing energy. These results are caused by maximum rotation capacities of column bases.
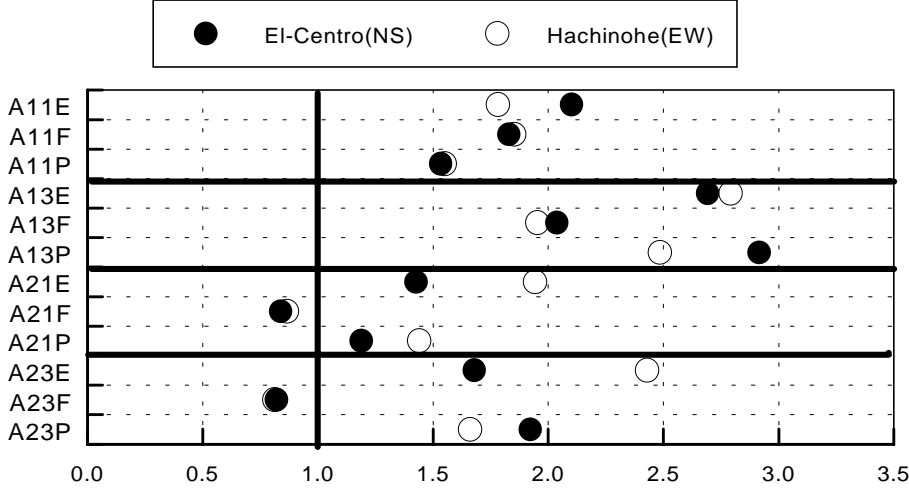

Fig.9 Ultimate Earthquake Resistance

CONCLUSION

From the analytical results, following conclusions were obtained.

1) The intensity of the damage concentration at the structural element of the 1st story in the frames with exposed type column bases were much less than that of the frames with pin type column bases and the frames with fixed column bases.

2) Ultimate earthquake resistance of the frames with exposed type column bases is similar to that of the frames with pin type column bases.

3) Ultimate earthquake resistance of the frames with exposed type column bases is much better than that of the frames with fixed column bases.

4) Absorbed energy by the exposed type column bases was much less than other structural elements.

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