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ULTIMATE BEHAVIOR OF HIGH-STRENGTH AND HIGH-QUALITY STEEL FRAME ANALYZED BY CANTILEVER ELEMENT

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ABSTRACT

This is the study on the collapse of frame against extraordinary strong ground motion not considered in the design code. The entire collapse behaviors of multi-story steel frames subjected to monotonic horizontal load and extremely strong ground motion are numerically analyzed by the use of the cantilever element (CE-Model) which was proposed by the author. As an important earthquake resistant factor, the energy absorbing capacity of frame until entire collapse (E_A) is calculated. The absorbed energy in seismic response of frame under extremely strong ground motion is compared with the value of E_A and it is shown that frames collapse dynamically when the absorbed energy of frame reaches to the energy absorbing capacity (E_A).

KEYWORDS

multi-story steel frame, high-strength steel, beam-column, numerical analysis, cantilever element model, seismic response analysis, entire collapse of frame, dynamic collapse of frame, energy absorbing capacity

1. INTRODUCTION

It is difficult to predict precisely the upper limit of earthquake intensity and we need to carry out earthquake resistant design against not only a major earthquake considered in the design code but also the extraordinary strong ground motion not considered in the design code. To execute the earthquake resistant design mentioned above the ultimate behavior of structure until it collapses entirely need to be investigated. We have never studied on the entire collapse behavior by laboratory test nor theoretical analysis. We have only observed the catastrophic behavior of buildings after strong shock of earthquake. In this study the entire collapse behaviors of high-strength steel frame and mild steel frame under monotonic load are analyzed numerically using the CE-Model proposed by the author.[4), 5)] From the calculation the energy absorbing capacity of frame until entire collapse (E_A), which changes according to the degrading behavior of frame restoring force especially the material properties of steel, is derived as an important earthquake resistant factor of frame. It is ascertained that the energy absorbing capacity corresponds to the absorbed energy in seismic response of frame when it collapses dynamically under extraordinary strong ground motion.

2. ANALYSIS BY CANTILEVER ELEMENT MODEL (CE-MODEL)

Numerical analysis of CE-Model used in this study is derived from the following assumptions.

i) CE-Model is a cantilever steel member and subjected to three incremental loads (ΔFx, ΔFz, ΔF_T) at the free end. (Fig.1)

ii) CE-Model deforms only in X-Z plane and the shear stress and shear strain of member are neglected.

iii) The section of member is expressed by the two-flange section. (Fig.2) The area of each flange is a/2 and the distance between the two flanges is 2d, where "a" and "d" express the sectional area and the radius of gyration of member respectively.

iv) The incremental normal stress ($\Delta\sigma$) in the flanges distributes linearly along the axis of member. (Fig.3)

v) The incremental normal strain ($\Delta\varepsilon$) in the flanges is divided into the incremental elastic strain ($\Delta\varepsilon_E$) and incremental plastic strains ($\Delta\varepsilon_{PA}$, $\Delta\varepsilon_{PB}$). There are the relations between the incremental elastic strain and incremental plastic strain in each flange as shown in Eqs.(1).

$$\Delta\varepsilon_{PA}=(E/E_{TA}-1)(1-Z/Z_A)\Delta\varepsilon_E \quad \text{at } x=d,\ Z_A>Z>0$$
$$\Delta\varepsilon_{PB}=(E/E_{TB}-1)(1-Z/Z_B)\Delta\varepsilon_E \quad \text{at } x=-d,\ Z_B>Z>0 \qquad (1)$$

in which

$\Delta\varepsilon_E$: incremental elastic strain whose distribution is proportional to the incremental stress,

$\Delta\varepsilon_{PA}$: incremental plastic strain in the flange of x=d,

$\Delta\varepsilon_{PB}$: incremental plastic strain in the flange of x=-d,

E: modulus of elasticity,

E_{TA}, E_{TB}: ($=\Delta\sigma/\Delta\varepsilon$) tangent moduli of stress strain relation in the flanges x=d and x=-d respectively,

Z_A, Z_B: length of plastic zone in the flanges of x=d and x=-d respectively. (Fig.3)

vi) The relation between the incremental normal strain ($\Delta\varepsilon$) and incremental deformations at the center of section ($\Delta\xi$, $\Delta\zeta$) is given by Eq.(2).

$$\Delta\varepsilon=\Delta\zeta'+\xi'\Delta\xi'-\Delta\xi''\,x \qquad (2)$$

where

$\Delta\xi$, $\Delta\zeta$: incremental displacements at the center of section Z in X-direction and Z-direction respectively, (Fig.1)

[']: differentiation with Z,

Δ: increment.

vii) Only in the fixed-end section (Z=0), the incremental stress ($\Delta\sigma$) and strain ($\Delta\varepsilon$) satisfy the assumed stress-strain relation.

Based on the assumptions, the relation between incremental loads (ΔFx, ΔFz, ΔF_T) and incremental deformations (ΔDx, ΔDz, ΔD_T) of the column-end is derived as follows.

$$\{\Delta f\}=[k]\{\Delta d\} \qquad (3)$$

in which $\{\Delta d\}^T=[\Delta Dx/L\ \ \Delta Dz/L\ \ \Delta D_T]$

$\{\Delta f\}^T=[\Delta Fx/Ny\ \ \Delta Fz/Ny\ \ \Delta F_T/NyL]$

Ny: yield axial force of member.

In Eq.(3), [k] is the stiffness matrix of CE-Model which is comparatively simple to calculate because [k] is expressed only with the summation of (ΔDx, ΔDz, ΔD_T), E_T (= E_{TA} or E_{TB}) and Fz. This is the reason why CE-Model is used in this collapse analysis of frame which generally requires huge computer calculation.

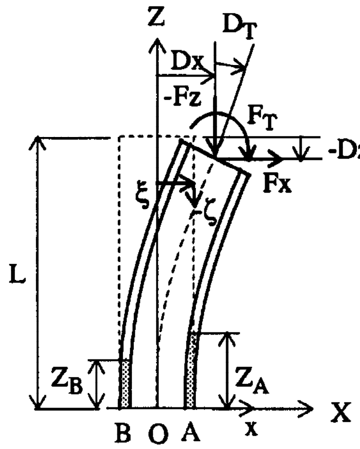

Fig.1 CE-Model

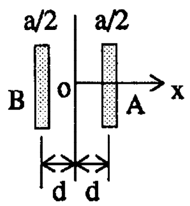

Fig.2 Section of CE-Model (Two-flange section)

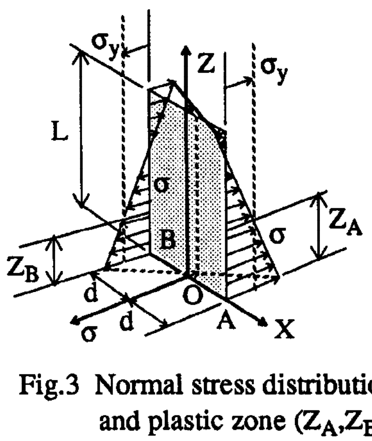

Fig.3 Normal stress distribution (σ) and plastic zone (Z_A,Z_B)

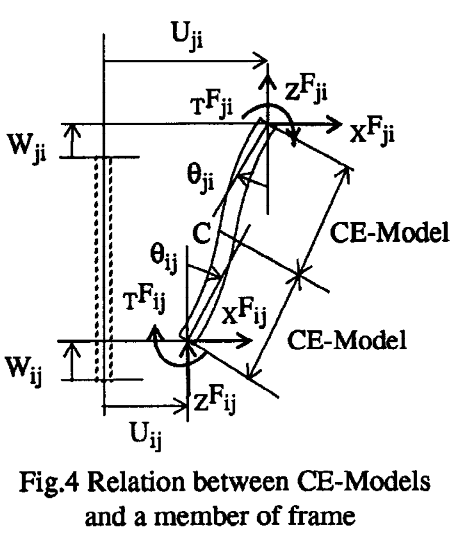

Fig.4 Relation between CE-Models and a member of frame

To apply CE-Model to frame analysis easily, the stiffness matrix of a member is derived. The stiffness matrix [K] of a member can be derived by coupling two CE-Models as shown in Fig.4. Using the equilibrium conditions and the continuity conditions at the center of member the stiffness matrix of a member can be obtained.

$$\{\Delta F\}=[K]\{\Delta D\} \qquad (4)$$

in which

$\{\Delta F\}^T=\{\Delta_X F_{ij}\ \Delta_Z F_{ij}\ \Delta_T F_{ij}\ \Delta_X F_{ji}\ \Delta_Z F_{ji}\ \Delta_T F_{ji}\}$,

$\{\Delta D\}^T=\{\Delta U_{ij}\ \Delta W_{ij}\ \Delta\theta_{ij}\ \Delta U_{ji}\ \Delta W_{ji}\ \Delta\theta_{ji}\}$.

[K] is the six dimensional stiffness matrix of a member expressed by the stiffness matrix of CE-Model [k] in Eq.(3).

3. STRESS-STRAIN RELATION OF STEEL

Mechanical properties of the high-strength and high-quality steel (HT780) and the widely used mild steel (SS400) are shown in Table 1.[1] High-strength steel generally shows the round-house stress-strain relation which is different from that of SS400 steel. The stress-strain relation of HT780 steel is also the round-house type but the yield stress ratio (σy/σu) of HT780 steel is small enough. From this reason the stress-strain relation of both HT780 and SS400 steels are expressed by the same type model as shown in Fig.5.

It is also assumed in this study that crack of steel starts at the breaking strain (ε_b) explained in Fig.5. This crack development is only assumption in numerical analysis and it is not based on any theory nor test data. It is also assumed that immediately after the crack starts in a member, the restoring force of it is lost completely. Accordingly if the crack develops in a member of frame the restoring force of it is redistributed to the adjacent members.

4. CALCULATION OF BEAM-COLUMN MEMBERS

To show the degrading behavior of each member composing multi-story steel frame, cantilever columns subjected to constant axial load (Fz) and lateral load (Fx) shown in Fig.6 are calculated by the CE-Model.

There are the calculated results in Fig.7, which is the load deformation relations of beam-column member under monotonic loading. There is the clear difference of plastic deformation capacity and degrading of restoring force between the two steels. From this figure we can see that entire collapse of frame changes according to the complicated restoring force characteristics of each member. It is also shown that the analysis method of entire collapse of frame need to calculate the complicated degrading behavior of all members composing the frame.

5. DESIGN OF FRAMES

Four steel frames have been calculated to investigate entire collapse of frame. They are 15-story frames (HT15-Frame, SS15-Frame) and 6-story frames (HT6-Frame, SS6-Frame) whose loads and design conditions are shown in Table 2, Table 3 and Fig.8. The columns of HT15-Frame and HT6-Frame are designed by high-strength steel (HT780) and the beams of them are widely used steel (SS400). SS15-Frame and SS6-Frame are designed only by SS400 steel.

They are designed under the following conditions.[3]

i) The weight and horizontal strength of the frames are the same among them whose base shear ratio calculated by the plastic analysis method is 0.2. The distribution of shear strength (Qi) is given based on the Japanese design code as shown in Table 2-3.

ii) Every frame is designed as the beam-yield-type frame. Every column over design factor (COF) is 1.2. (Table 2-3)

iii) The section of each member satisfies the following relations.

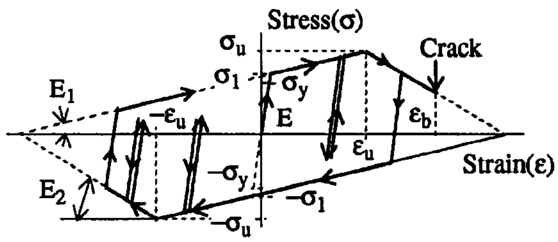

Fig.5 Hysteretic stress-strain model

Table 1 Mechanical properties of steel

steel	σ_y	σ_u	YR	ε_u	ε_b	E_1/E	E_2/E
SS400	3.0	4.6	0.65	0.15	0.40	1/500	-1/100
HT780	6.7	8.3	0.81	0.075	0.30	1/300	-1/100

σ_y: yield stress(tf/cm²), σ_u: ultimate strength(tf/cm²), ε_u: strain at ultimate strength

ε_b: fracture strain, YR: (=σ_y /σ_u) yield stress ratio,

E, E_1, E_2: modulus of elasticity, strain hardening and degrading respectively.

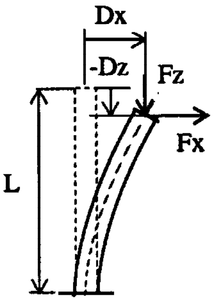

Fig.6 Loads and deformations of cantilever column (L/d=15.0)

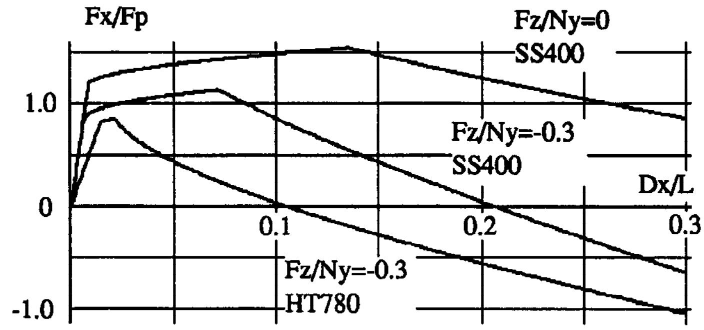

Fig.7 Load deformation relation of cantilever column under constant axial load and monotonic lateral load ($Fp=Mp/L$, Mp: full plastic moment, Ny: yield axial force)

$d^2/a = 1.0$: columns
$d^2/a = 4.0$: beams (5.1)

in which d: radius of gyration, a: sectional area of member.

The designed frames under the conditions mentioned above are shown in Table 2-3 and the natural frequencies of them are also given in Table 4.

6. COLLAPSE OF FRAMES UNDER MONOTONIC LOAD

Loading conditions of frame

Entire collapse of the frames are calculated under monotonic loading. The loads of the frames are the gravity load and the monotonic horizontal load which work only on every beam-column connection. The horizontal load is the proportional load whose proportion is the same as that used in the design of frames shown in Table 2-3.

Load deformation relations (Fig.9)

Fig.9. shows the relation between horizontal load (ρ) and horizontal displacement (Ui, i: number of floor) of the frames. In these figures ρ is the ratio of horizontal load to the ultimate strength calculated by the plastic analysis of frame.

We can see in these figures not only the difference of plastic deformation capacity but also the drastic difference of degrading frame restoring force until entire collapse among the frames. This is the reason why we need to analyze the entire collapse of frame to carry out the earthquake resistant design against extraordinary strong

Table 2 Design conditions of 15-story frames

Floor	Wi	Qi/Q_1	Mp (column) I	Mp (column) II	Mp (beam)
1	75	1.00	9.09	18.18	15.01
2	"	1.05	8.92	17.84	14.69
3	"	1.10	8.71	17.42	14.30
4	"	1.15	8.45	16.90	13.83
5	"	1.21	8.14	16.29	13.28
6	"	1.27	7.79	15.59	12.66
7	"	1.34	7.39	14.79	11.95
8	"	1.41	6.95	13.90	11.17
9	"	1.48	6.45	12.90	10.29
10	"	1.57	5.90	11.80	9.33
11	"	1.67	5.29	10.59	8.26
12	"	1.80	4.62	9.24	7.08
13	"	1.96	3.88	7.76	5.77
14	"	2.21	3.05	6.10	4.29
15	100	2.65	2.09	4.19	4.29

Wi: Weight of i-story (tf)
Qi: Shear strength of i-story
Mp: Full plastic moment of member ($\times 10^3$tf.cm)

Table 3 Design conditions of 6-story frames

Floor	Wi	Qi/Q_1	Mp (column) I	Mp (column) II	Mp (beam)
1	75	1.00	3.69	7.37	5.93
2	"	1.11	3.43	6.87	5.43
3	"	1.23	3.09	6.18	4.78
4	"	1.37	2.65	5.30	3.96
5	"	1.55	2.10	4.20	2.94
6	100	1.84	1.43	2.86	2.94

Table 4 Natural frequency of frames

Frame	ω_1	ω_2
HT15-Frame	1.92	4.91
SS15-Frame	2.57	6.53
HT6 -Frame	2.70	7.06
SS6 -Frame	3.55	9.74

ω_1, ω_2: Natural frequency of first mode and second mode respectively.

ground motion.

Shape of frame deformation (Fig.10)

In Fig.10 there are the shapes of deformed frame at the maximum load ($\rho=\rho_{max}$), at the full plastic load ($\rho=1$) and at the entire collapse ($\rho=0$). In these figures only the displacements of beam-column connection are drawn and the deformation of each member is not expressed. Although the horizontal strength of the frames and the proportion of horizontal load are the same among the frames, the remarkable difference of the deformed shape among them appears especially in the degrading state. The figures also show that the shape of deformed frame changes its mode of deformation as the deformation increases and the restoring force degrades.

Distribution of member deformation (Fig.11)

Deformations of all columns and beams of frame are in Figs.11 in which the thick lines drawn at the end of each member show the rotation (Dx/L) of its member. In the figures there are the deformations at the maximum load ($\rho=\rho_{max}$), at the full plastic load ($\rho=1$) and at the entire collapse ($\rho=0$).

The deformation of beam is larger enough than that of column and the deformation of beam is distributed almost uniformly in the frame at $\rho=\rho_{max}$ because all frames are designed as the beam-yield type. But the deformation of columns increases extremely after the maximum load ($\rho=\rho_{max}$) and the deformation of columns is concentrated only in a few columns. Considering the distribution of column deformation, it is clear that the degrading of frame restoring force is closely related to the concentration of plastic deformation in a few columns.

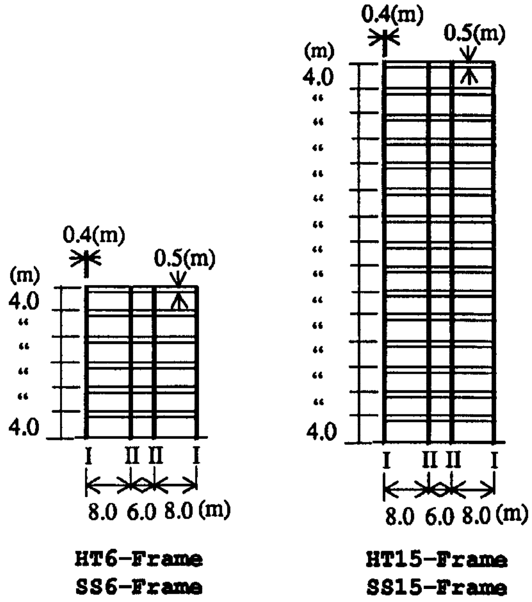

Fig.8 Dimensions of calculated frames

7. ENERGY ABSORBING CAPACITY OF FRAMES

Energy absorbing capacity (E_A) of the calculated frames are in Table 5. The values of E_{Am}, E_{A1} and E_A are the absorbed energy until the maximum load ($\rho=\rho_{max}$), the full plastic load ($\rho=1$) and the entire collapse ($\rho=0$). Ep ($=\Sigma\ w_i \cdot h_i$, w_i, h_i: the weight and height of i-floor) in the table means the potential energy of frame. We can see that the material properties of steel changes the energy absorbing capacity (E_A) significantly. It is also pointed out that the ratios of the absorbed energy (E_{Am}/E_A and E_{A1}/E_A) change according to the design conditions of frame. From this reason to analyze the ultimate state without the degrading behavior and the entire collapse of frame is not sufficient for the earthquake resistant design

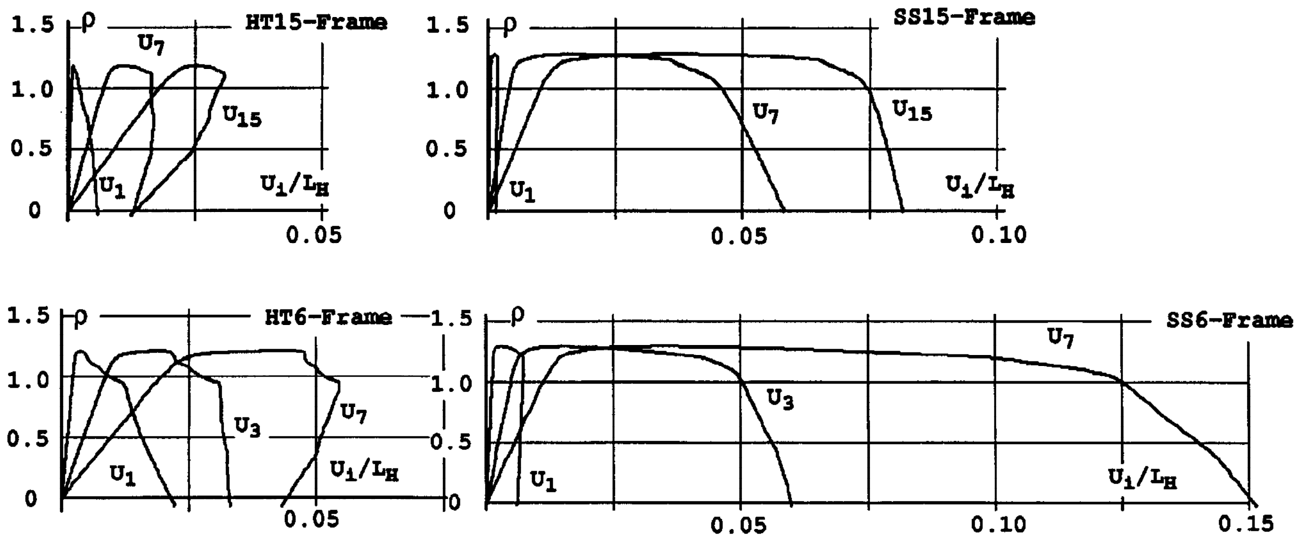

Fig.9 Horizontal load(ρ) and deformation (Ui) of frames under monotonic load (L_H:Height of frame)

Table 5 Energy absorbing capacity of frames

Frame	E_{Am}/E_P	E_{A1}/E_P	E_A/E_P
HT15-Frame	0.45	0.64	0.53
SS15-Frame	1.02	2.48	2.63
HT6 -Frame	0.90	1.32	1.37
SS6 -Frame	0.88	3.51	3.85

E_{Am}, E_{A1}, E_A :Absorbed energy until $\rho=\rho_{max}$, $\rho=1$ and $\rho=0$ respectively,
E_P :Potential energy of frame

against extraordinary strong ground motion.

8. *COLLAPSE OF FRAMES UNDER EXTREMELY STRONG GROUND MOTION*

Conditions of seismic response analysis

Earthquake response analysis is carried out by the use of CE-Model and the dynamic collapse of frame under extremely strong ground motion is calculated.

The conditions used in this analysis are as follows.

i) Mass of frame is concentrated in the beam-column connections. Rotational inertia is decided by assuming that the concentrated mass of frame distributes uniformly in the panel zone of connection.

ii) Viscous damping matrix of frame is given by a linear combination of the mass and stiffness matrices. The damping factors of the first mode and the second mode are 0.02.

iii) To give extremely strong ground motion whose duration is long enough to make frames collapse, an artificial ground motion is used in the analysis. The seismic response spectrum of it is shown in Fig.12.

iv) Step-by-step analysis procedure is used in the calculation. The size of increment is decided under the condition that the numerical error of the energy balance is always less

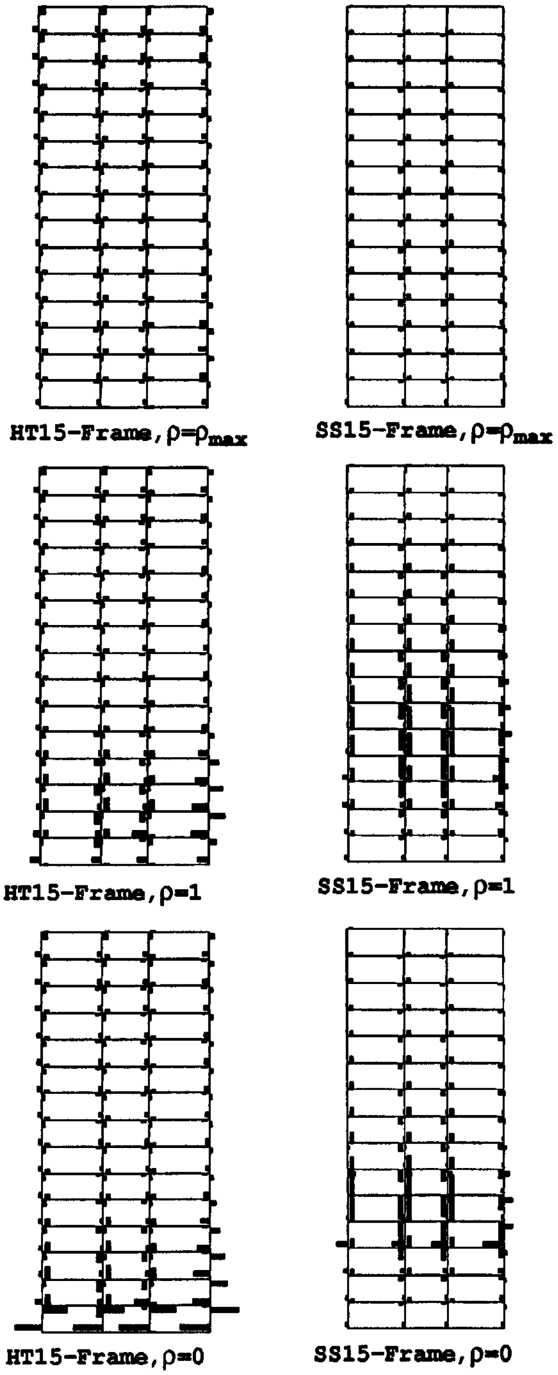

Fig.11 Distribution and concentration of member deformation in frame

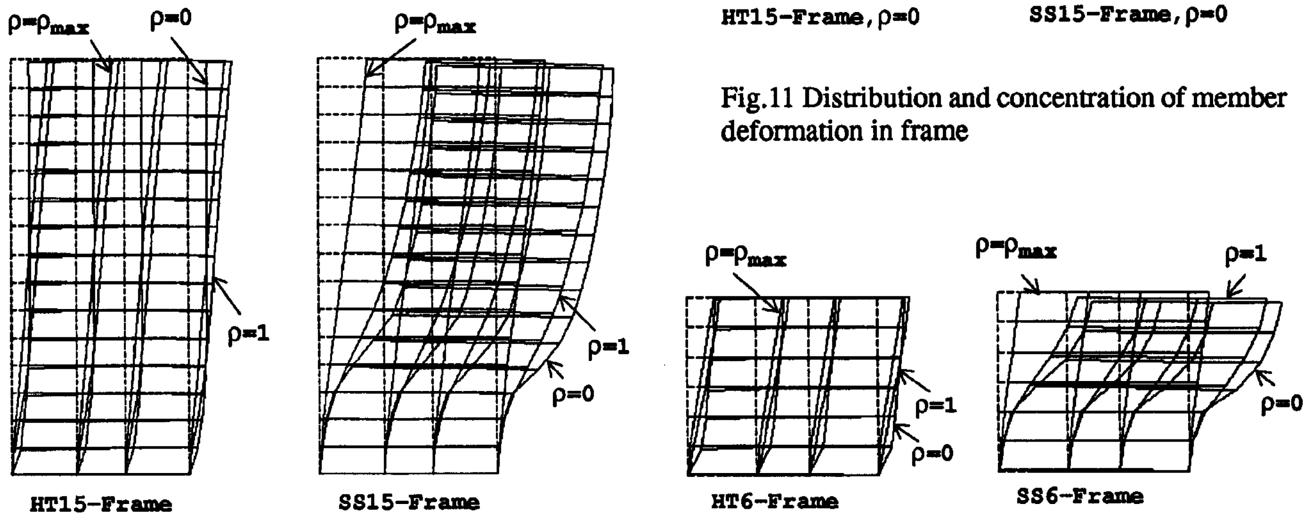

Fig.10 Deformation of frames under monotonic load (amplified by 3 times)

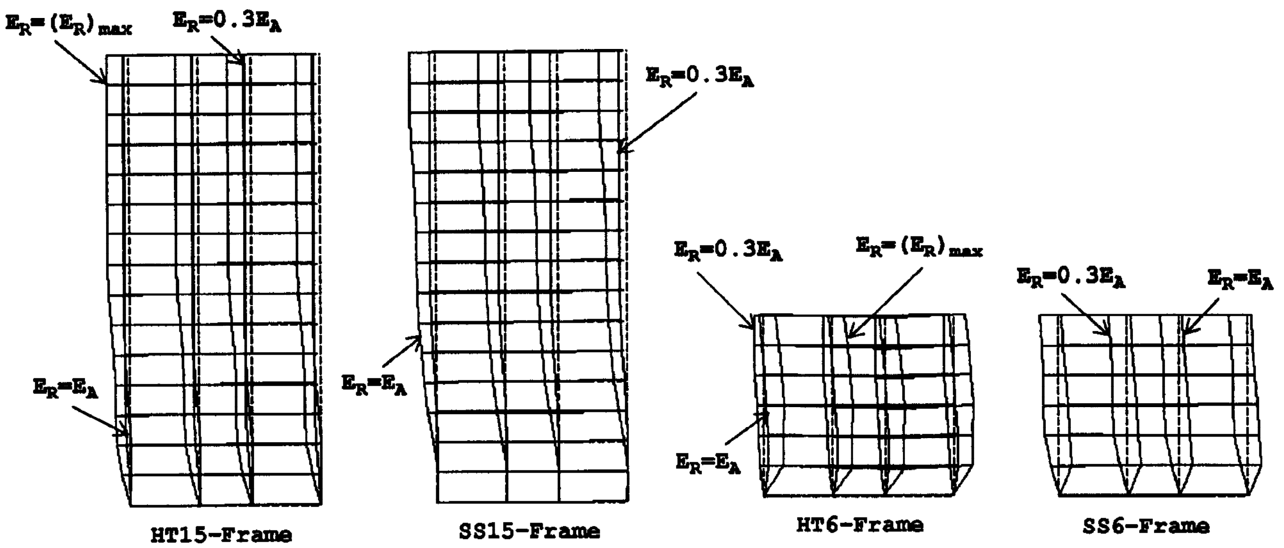

Fig.13 Deformation of frames under extremely strong ground motion (amplified by 3 times)

than 0.1% of the input energy. [2)]

Time history of response displacement (Figs.14)
In Fig.14 there are the time histories of horizontal and vertical response displacements (Ui, Wi) which are expressed by the thin and thick lines respectively. Fig.14 shows the seismic response of frame increases gradually and the frames collapse incrementally. These incremental collapse is not directly related to the degrading or loss of restoring force of frame because the seismic response of deformation is mainly decided by the input ground motion.
We are interested in the dynamic entire collapse under extraordinary strong ground motion. But it is difficult to decide the state and condition of dynamic collapse from the time history of seismic response shown in Fig.14.

Shape of deformed frame (Fig.13)
There are the shapes of frame deformation in Fig.13. The deformations in the figure are corresponding to the states when the absorbed energy of restoring force (E_R) is equal to $0.3E_A$, E_A and the maximum absorbed energy $(E_R)_{max}$.
The shape of frame deformation is quite different from that under the proportional monotonic load shown in

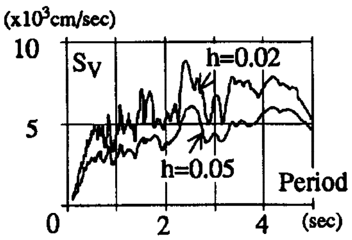

Fig.12 Seismic response spectrum (S_V) of artificial ground motion (h: damping factor)

Fig.10 and the deformation of frame is concentrated in the first story or in the lower stories. This concentration of frame deformation is closely related to the condition that the input ground motion is extremely strong.

9. ABSORBED ENERGY IN SEISMIC RESPONSE

The absorbed energy by restoring force (E_R) is calculated and the ratio of it to the energy absorbing capacity under monotonic load (E_A) is shown in Fig.15. There are remarkable differences of distribution and concentration of plastic frame deformation between under monotonic load and seismic load. (Fig.10, Fig.13) But we can see in Fig.15 that the horizontal or vertical displacement increases remarkably and the frames collapse dynamically when the absorbed energy (E_R) reaches to the energy absorbing capacity (E_A). From this reason we can say that the energy absorbing capacity (E_A) is one of the important factors to decide earthquake resistant capacity of frame under extremely strong ground motion.

10. CONCLUSIONS

Collapse of frame and effectiveness of CE-Model analysis
Entire collapse of frame is closely related to the degrading behavior of every beam and column composing the frame. Especially the drastic degrading of frame restoring force is significantly influenced by the failure of only a few columns. Accordingly the plastic deformation and degrading behavior of every beam and column member must be analyzed accurately enough to deal with the entire collapse of frame. (Fig.11)
The numerical analysis method of CE-Model proposed by the author can analyze the behavior mentioned above

and makes it possible to calculate the entire collapse of frame by comparatively simple calculation.

Collapse of frame under monotonic load and energy absorbing capacity (E_A)

In the process to the entire collapse of frame the frame deformation mode changes and the plastic deformation is concentrated in a few columns in spite of the proportional load.(Fig.10) Especially the plastic deformation of HT15-Frame and HT6-Frame, whose columns are designed by high-strength steel, is concentrated in the columns of the first story and the frame restoring force degrades drastically. (Fig.9)

The plastic deformation capacity and the drastic degrading of restoring force can be expressed clearly by the absorbed energy of frame until the maximum load (E_{Am}), the full plastic load (E_{A1}) and the entire collapse (E_A). (Table 5)

Collapse of frame under seismic load

The frame deformation under extremely strong ground motion, which is concentrated in the first story or lower stories, is quite different from that under monotonic proportional load. (Fig.13)

Absorbed energy in dynamic collapse (E_R) and energy absorbing capacity (E_A)

In seismic response of frame under extraordinary strong earthquake motion the horizontal and vertical deformation of frame increases drastically when the absorbed energy of frame (E_R) reaches to the energy absorbing capacity (E_A). (Fig.14-15) From this fact the energy absorbing capacity (E_A) is an important factor to decide earthquake resistant capacity of frame subjected to extremely strong ground motion.

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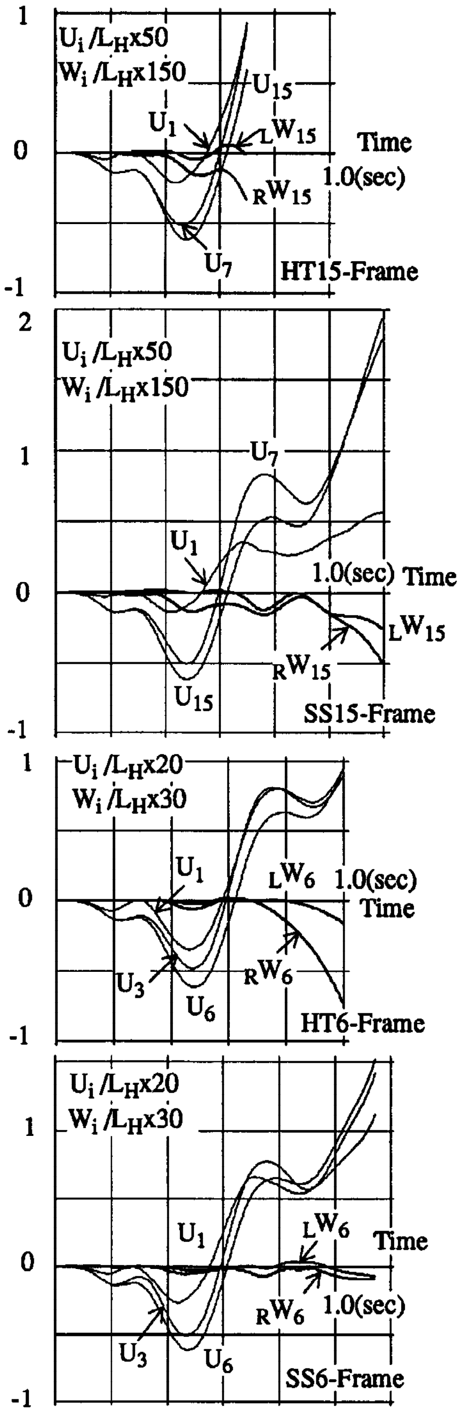

Fig.14 Time histories of seismic response displacement

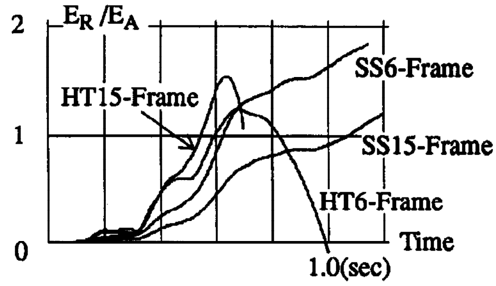

Fig.15 Time histories of the absorbed energy (E_R) against the energy absorbing capacity until entire collapse (E_A)