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ULTIMATE EARTHQUAKE RESISTANT CAPACITY OF CFT-FRAME

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SUMMARY

Seismic response and damage of concrete filled steel tube frame (CFT frame) under extremely strong ground motion are calculated by the numerical analysis method presented by one of the authors. The damage ratios of steel tube crack and local buckling of CFT column, which are closely related to the collapse of CFT frame, are obtained in relation with the distribution of story-shear strength, the column-over-design factor of frame and the concrete-to-tube strength ratio of CFT column. On the basis of the calculated damage ratios, the simple formula expressed by the design factors mentioned above is proposed to predict the ultimate earthquake resistant capacity of multi-story CFT frame.

1. INTRODUCTION

It is well known that the concrete filled steel tube column (CFT column) is the ductile and useful member as earthquake resistant element. But in some cases under strong seismic load the CFT-column fractures by the crack of steel tube. The fracture of CFT column is brittle and works to collapse the whole CFT frame under strong ground motion. From this reason the ultimate earthquake resistant capacity is investigated in this study in relation with the CFT column fracture caused by the crack of steel tube.

One of the authors proposed the collapse analysis method of CFT frame under extremely strong ground motion by introducing the steel tube crack condition and the damage ratio equation of CFT column[1)-2)]. By the use of this numerical analysis method many calculations of the seismic response and the dynamic collapse of CFT frame under the recorded strong ground motion (JMA-KOBE NS & UD) are carried out and the simple formula to predict the ultimate earthquake resistant capacity is obtained.

2. DAMAGE RATIO OF CFT-FRAME AND CFT-COLUMN

2.1 Dynamic collapse of CFT frame

The dynamic collapse of multi-story CFT frames which are subjected to extremely strong ground motion

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has been carried out. The CFT frames are composed of CFT columns and H-section beams.
From the calculation results the following collapse behaviors and falling down process are obtained[1)-2)].

i) When CFT frame is subjected to strong ground motion, at first the steel tube of CFT column buckles locally near the column end. The local buckling is related closely to the crack of steel tube.

ii) After the first crack of CFT column, the steel tube crack appears successively in other CFT columns of the same story. At the same time the steel tubes of CFT column in other stories also buckle locally.

iii) When the steel tube cracks and the restoring force of CFT column is lost suddenly, the CFT frame begins to fall down locally. After the local failure of frame in the lower stories the collapse of CFT frame develops repeating the collision between the upper and the lower rigid panels of beam-column connection and the shape of response deformation of frame changes complicatedly with time.

2.2 Damage ratio of CFT frame and CFT column

As mentioned above, the dynamic collapse of CFT frame under strong seismic load is closely related to the crack of CFT column. From this reason the damage ratio of CFT frame under seismic load can be expressed approximately by the damage ratio of CFT column defined by the steel tube crack.
The crack of H-section beam also effects on the dynamic collapse of CFT frame under strong seismic load. But the H-section beam crack does not effect so significantly on the dynamic collapse of CFT frame as the crack of CFT column does. From this reason the effect of CFT column crack on the dynamic collapse of CFT frame is mainly investigated in this study.

2.3 Damage ratio of CFT column

The damage ratio of CFT column is obtained to express approximately the damage ratio of CFT frame under strong seismic load.
The dynamic and repeated loading test of CFT column was carried out[3)] and the steel tube crack of CFT column was investigated[4)-5)]. On the basis of the test results it is pointed out that the steel tube of CFT column cracks when the accumulated plastic strain of steel tube becomes to be the critical value ($\alpha\varepsilon_f$). From this result the crack condition of CFT column is expressed by Eq.(1).

$$\Sigma\varepsilon_{TC}+\Sigma\varepsilon_T = \alpha\varepsilon_f \qquad (1)$$

in which ε_T is the plastic tension strain of steel tube in the tension stress side and ε_{TC} is also the plastic tension strain due to the local buckling deformation of steel tube in the compression stress side as explained in Fig.1. According to Eq.(1) the damage ratio of steel tube crack (D_{cr}) can be expressed by Eq.(2).

$$D_{cr} = (\Sigma\varepsilon_{TC}+\Sigma\varepsilon_T)/\ \alpha\varepsilon_f \qquad (2)$$

As shown in Eq.(1) the local buckling of steel tube is closely related to the steel tube crack. From this reason the condition for steel tube to buckle locally is obtained on the basis of the upper bound theorem of the limit analysis[6)]. The damage ratio of local buckling (D_{lb}) is decided by the use of critical deformation (δ_{lb}) which corresponds to the CFT column deformation for the steel tube to buckle locally.

$$D_{lb} = (\delta_{PC} - \delta_{PT})/\ \delta_{lb} \qquad (3)$$

in which ($\delta_{PC} - \delta_{PT}$) is the amplitude of plastic deformation of CFT column.

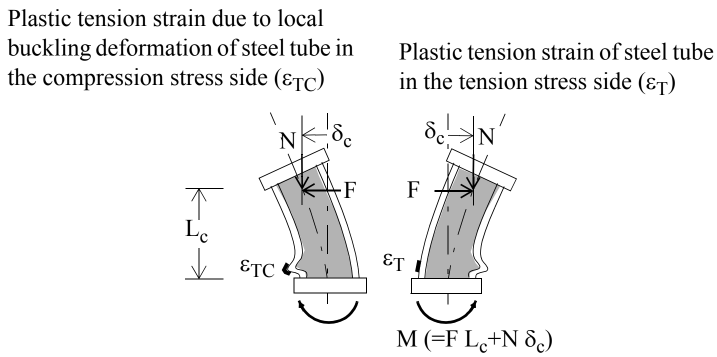


Fig.1 Local buckling deformation and tension strain of steel tube

3. DESIGN OF CFT-FRAME

3.1 Design conditions of CFT frame

Multi-story CFT plane frames analyzed in the next section are composed with CFT columns and H-section beams and they are 15-story 3-bay frames (F15-3 frame) and 7-story 3-bay frames (F7-3 frame). The height of story is 4.0 m and the lengths of outer span and inner span are 8.0 m and 6.0 m respectively. The weight of each story is 2000KN. These frames are designed under the following conditions.

i) The base shear strengths of 15-story 3-bay frames (F15-3 frame) and 7-story 3-bay frames (F7-3 frame) are 0.25W (W: weight of frame) and 0.40W respectively. The frame strength is calculated by the limit analysis method.

ii) The story-shear strength ratio of every story (C_i) is decided strictly according to the assumed distribution explained in the following section.

iii) The column-over-design factor (r_c) of every beam-column connection except the highest story is the same as mentioned in the following section.

iv) The strength ratio of filled concrete to steel tube ρ (=$\sigma_c A_c/\sigma_u A_s$, A_c, A_s: sectional areas of concrete and steel tube respectively) effects on the restoring force characteristics of CFT column[3)-5)]. From this reason the concrete-to-tube strength ratios of all CFT columns are assumed to be the same as mentioned in the following section.

v) Every H-section beam of frame satisfies the relation of r_x^2/A=4.0 (r_x: radius of gyration, A: sectional area).

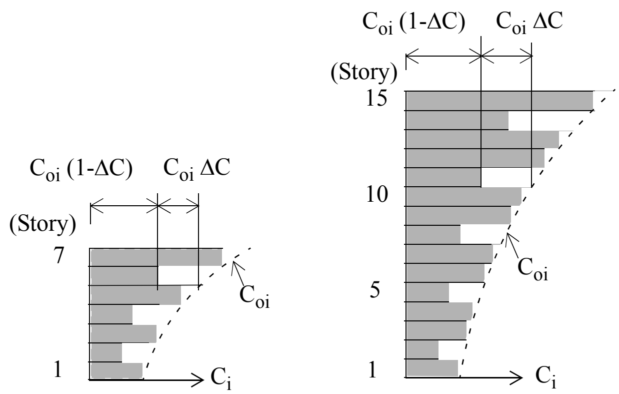
vi) The yield stress (σ_y) and tensile strength (σ_u) of steel tube and H-section beam are σ_y=340N/mm^2, σ_u=440N/mm^2. There are three kinds compression strength of concrete (σ_c= 30N/mm^2, 60N/mm^2, 120N/mm^2).

3.2 Design factors of CFT frame

Three design factors which are closely related to the dynamic collapse and the ultimate earthquake resistant capacity of CFT frame are considered in the design of CFT frame. They are the distribution of the story-shear strength, the column-over-design factor and the strength ratio of filled concrete to steel tube. These design factors of CFT frame are given as follows.

The distribution of the story-shear strength ratio is expressed by $C_{oi}(1\text{-}\Delta C)$ (i: number of story) in which C_{oi} means the basic distribution of story-shear strength ratio given in the Japanese design code and ΔC is the

deviation ratio from the basic value (C_{oi}). There are three kinds of deviation distribution according to the number of story whose shear-strength ratio deviates from the basic value (C_{oi}). We call the three kinds of CFT frame as A-Frame, B-Frame and C-Frame whose number of story (i) deviated from the basic value (C_{oi}) is given by i=n j+m (j=0, 1, 2,...) explained in Fig.2 and Table-1.
The CFT frames are designed under the condition of different story-shear distribution but there is no difference of the ultimate horizontal strength among them. In the decision of CFT frame strength, the ultimate horizontal strength of CFT frame under the proportional horizontal load defined by the basic distribution value (C_{oi}) is calculated based on the limit analysis.



7A-Frame
(7-story 3-bay frame, m=2, n=2)

15A-Frame
(15-story 3-bay frame, m=2, n=3)

Fig.2 Assumed distribution of story-shear strength ratio

Table-1 Parameters to define story-shear strength distribution

Name	Frame	m	n
7A-Frame	7-story 3-bay	2	2
7B-Frame		1	2
7C-Frame		1	3
15A-Frame	15-story 3-bay	2	3
15B-Frame		1	3
15C-Frame		1	7

In addition to the story-shear strength ratio, the following values are used to design the CFT frames whose column-over-design factor (r_c) and concrete-to-tube strength ratio (ρ) are different among them.

r_c =1.2, 1.5, 2.0

D/t=63, 93, 153

ΔC=0.0, 0.1, 0.3, 0.5, 0.7

The diameter to thickness ratios of steel tube (D/t=63, 93, 153) are corresponding respectively to the concrete-to-tube strength ratios (ρ) as follows.

ρ =1.0, 1.5, 2.5 (in case of σ_c= 30N/mm^2)

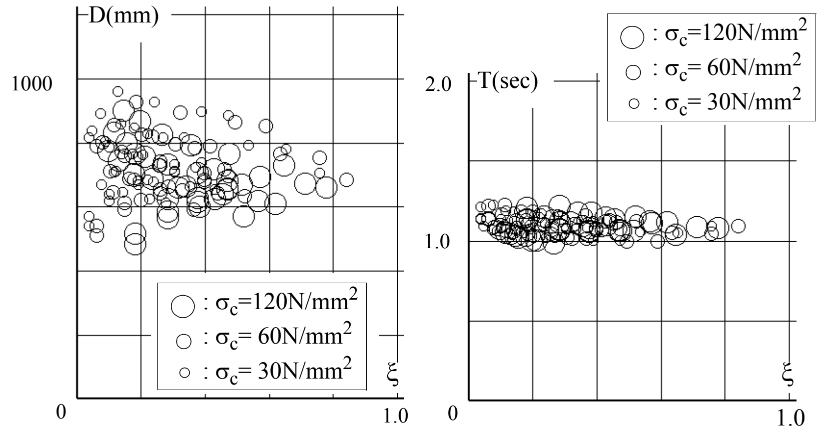
ρ =2.0, 3.0, 5.0 (in case of σ_c= 60N/mm^2)

ρ =4.0, 6.0, 10.0 (in case of σ_c=120N/mm^2)

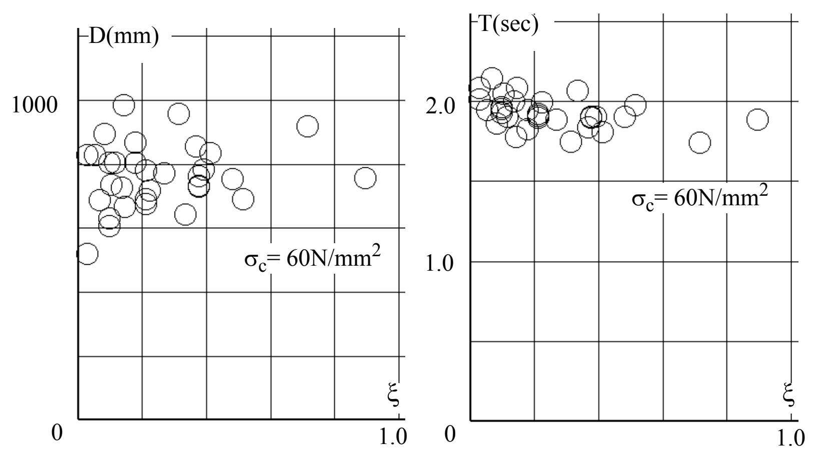
3.3 Designed CFT frames

Multi-story CFT frames are designed on the basis of the conditions mentioned above. In the design of CFT

frame it is also assumed that any section of steel tube is available. All designed frames are explained in Fig.3. They show the natural period of CFT frame (T) and the diameter of CFT column of the first story (D). The abscissa of the figures are the earthquake resistant parameter introduced in this study and explained later in Eq.(5).
Each CFT frame is designed under the quite different conditions. But the natural periods of all CFT frames are not different significantly among them and the size of CFT column member of each frame is realistic value. From these results we can also see that the design conditions of CFT frame introduced in this study is useful.



7-story 3-span frame (7A-Frame, 7B-Frame, 7C-Frame)



15-story 3-span frame (15A-Frame, 15B-Frame, 15C-Frame)

Fig.3 Steel tube diameter and natural period of designed CFT frame

4. SEISMIC RESPONSE ANALYSIS OF CFT-FRAME

4.1 Multi-story frame model

In the analysis of CFT frame, the multi-story plane frame is assumed to be composed of the rigid panel zones of beam-column connection and the axially elastic members with elastic-plastic hinges at the both ends as explained in Fig.4. The mass of frame is concentrated in every panel zone and distributed uniformly in it. Accordingly the deformation of frame can be expressed only by the rotation (θ_i, i: number of panel zone), the horizontal displacement (u_i) and the vertical displacement (w_i) of every rigid panel zone.

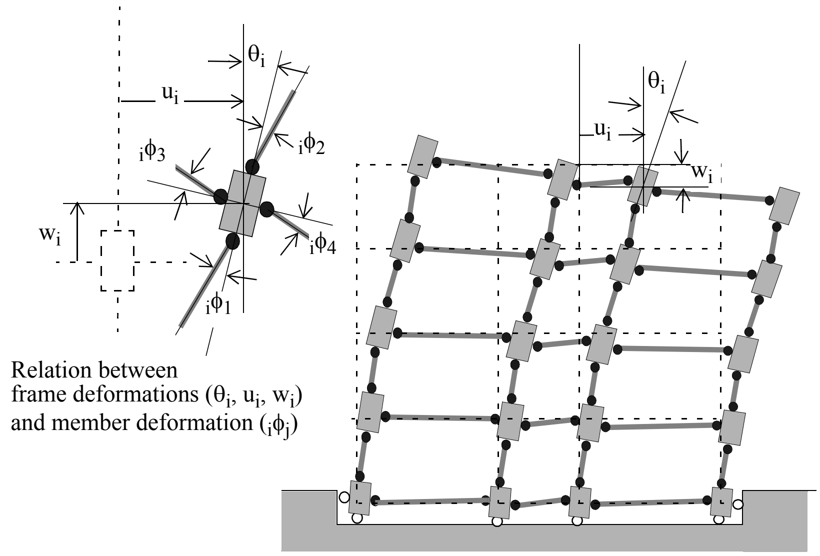


Fig.4 CFT frame model for numerical analysis

4.2 Restoring force model of member

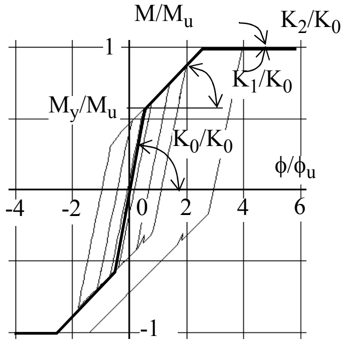
The restoring force model of elastic-plastic hinge is obtained on the basis of the dynamic loading test of CFT column[3)-5)]. According to the test results the non-dimensional restoring force (M/M_u) of CFT column until the local buckling of steel tube is approximated by the Tri-linear model whose skeleton curve is defined in Fig.5 and after the local buckling of steel tube it is expressed by the modified Clough model[7)] whose skeleton curve is shown in Fig.6. The stiffness ratios in the plastic range in Fig.5 and Fig.6 are assumed as K_1/K_0=0.2, K_2/K_0=0.001, $K_r=K_0/\mu_m^{0.5}$ (μ_m: the maximum ductility factor) in comparison with the test results[3)-5)].

The restoring force model is defined by the non-dimensional restoring force (M/M_u) in which the ultimate bending strength (M_u) changes according to the varying axial force of CFT column at every instance. By the use of the non-dimensional restoring force (M/M_u), the restoring force model is also effective for the CFT column under varying axial force.

The H-section beam of multi-story frame is also expressed by the same model of CFT column which is the elastic member with the elastic-plastic hinges at both ends as shown in Fig.4. The restoring force of the elastic-plastic hinge is decided by the Tri-linear model shown in Fig.7 in which the restoring force characteristics are given by the full plastic moment (M_p) and the ultimate bending strength (${}_bM_u$) of H-section beam. The strain hardening behavior of H-section beam effects on the seismic response and collapse of CFT frame under strong ground motion. Accordingly the strain hardening of H-section beam in the model should not be neglected. It is given by the value K_1 shown in Eq.(4) which is obtained by assuming the H-section beam can be approximated by the two-flange section member.

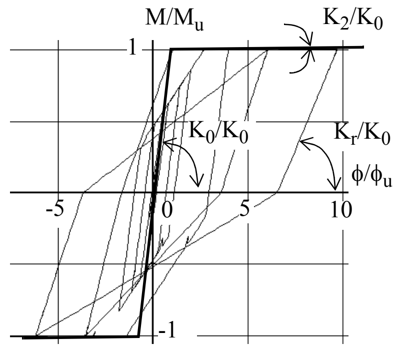
$$\frac{K_1}{K_0} = \frac{1/y-1}{1.5y(1-y)(1+u)-1} \tag{4}$$

in which $y(=\sigma_y/\sigma_u)$ and $u(=\varepsilon_u/\varepsilon_y)$ mean the yield stress ratio and the ultimate tensile strain ratio respectively.



(K_1/K_0=0.2, K_2/K_0=0.001)

Fig.5 Tri-linear model and its skeleton curve of CFT column



(K_2/K_0=0.001, K_r/K_0:Unloading stiffness)

Fig.6 Clough model and its skeleton curve of CFT column

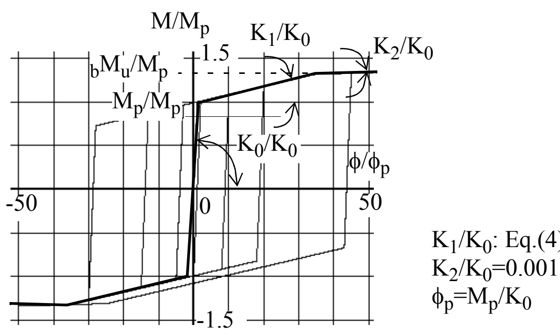


Fig.7 Tri-linear model and its skeleton curve of H-section beam

4.3 Ground motion

Seismic response of CFT frame under the strong ground motion JMA-KOBE NS & UD(1995) recorded in Kobe is calculated. In order to investigate the effect of the characteristics of ground motion, the artificial ground motion (ART NS & UD) whose spectrum intensity (SI) is the same as that of JMA-KOBE NS & UD(1995) is also used as input motion. The spectral accelerations are shown in Fig.8 comparing with each others.

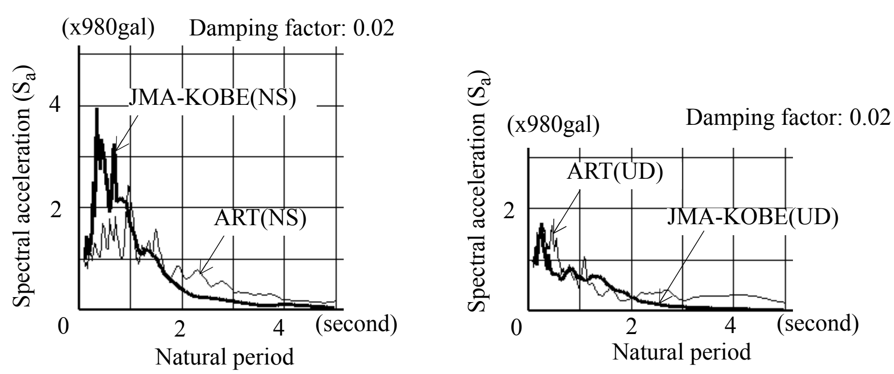


Fig.8 Seismic response spectra of the artificial ground motion (ART NS & UD) and the recorded strong ground motion (JMA-KOBE NS & UD)

5. ULTIMATE EARTHQUAKE RESISTANT CAPACITY

5.1 Damage concentration and distribution of CFT frame

The damage distributions of CFT frame obtained by the calculated seismic response are shown in Fig.9 and Fig.10. The damages of CFT frame shown in the figures are the maximum rotation of elastic-plastic hinge (ϕ_m), the maximum damage ratio of local buckling of CFT column (D_{lb}) and the maximum damage ratio of steel tube crack (D_{cr}). The damage and the damage ratio are expressed by the thick lines perpendicular to the axis of column and beam. The numerical values in the figures explain the damage or damage ratio in the first story and in the highest story.

The CFT frames shown in Fig.9 and Fig.10 are designed under quite different conditions of the column-over-design factor, the strength ratio of filled concrete to steel tube and the story-shear strength distribution. But it is shown that the crack damage ratio of every CFT frame is concentrated only in one story in account of the different design conditions of it. The maximum crack damage ratio concentrated only in one story is closely related to the whole collapse of CFT frame under strong seismic load. From this reason the maximum crack damage ratio expressed by $(D_{cr})_m$ is discussed in relation with design factors of CFT frame in the following sections.

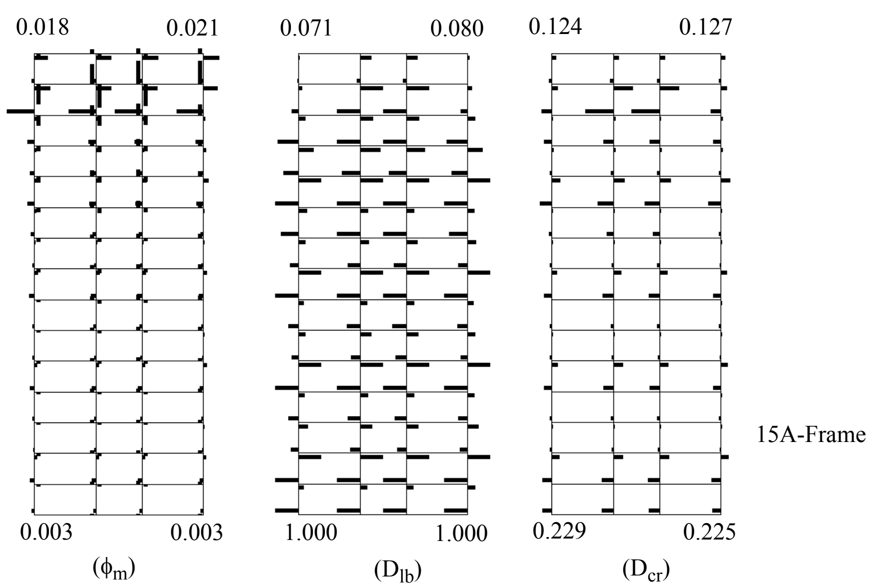


Fig.9(A) Damages of 15-story CFT frame under strong ground motion (JMA-KOBE NS & UD)
(ΔC=0.3, ρ=3.0 (D/t=93), r_c=1.5, σ_c=60N/mm^2)

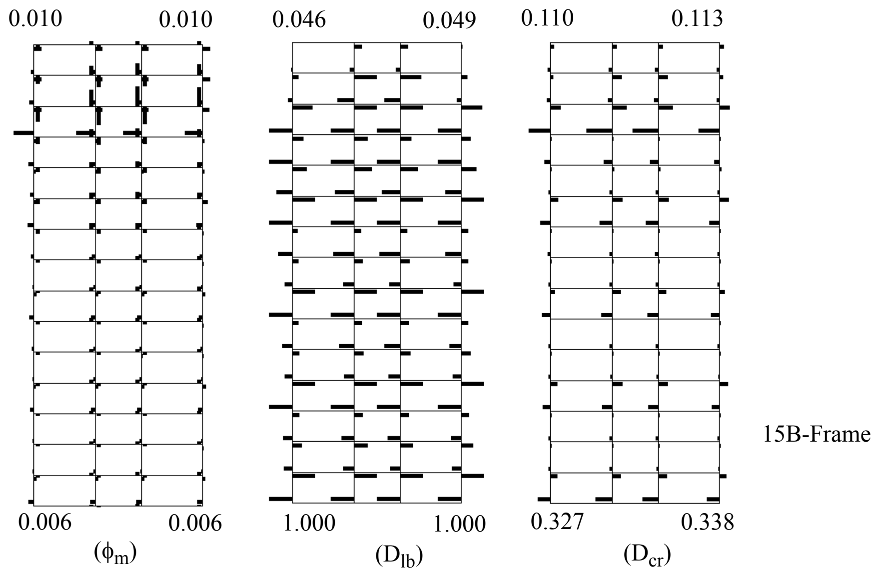


Fig.9(B) Damages of 15-story CFT frame under strong ground motion (JMA-KOBE NS & UD)
(ΔC=0.3, ρ=3.0 (D/t=93), r_c=1.5, σ_c=60N/mm^2)

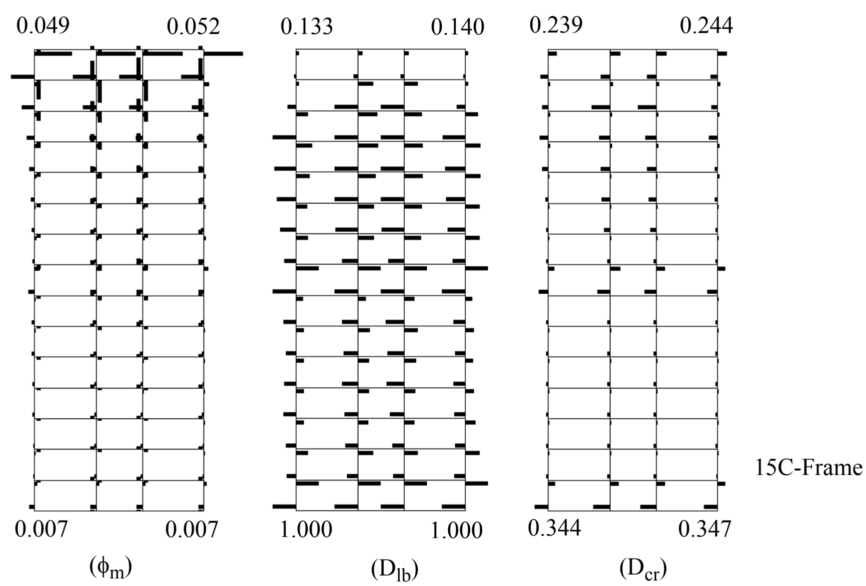


Fig.9(C) Damages of 15-story CFT frame under strong ground motion (JMA-KOBE NS & UD)
(ΔC=0.3, ρ=3.0 (D/t=93), r_c=1.5, σ_c=60N/mm^2)

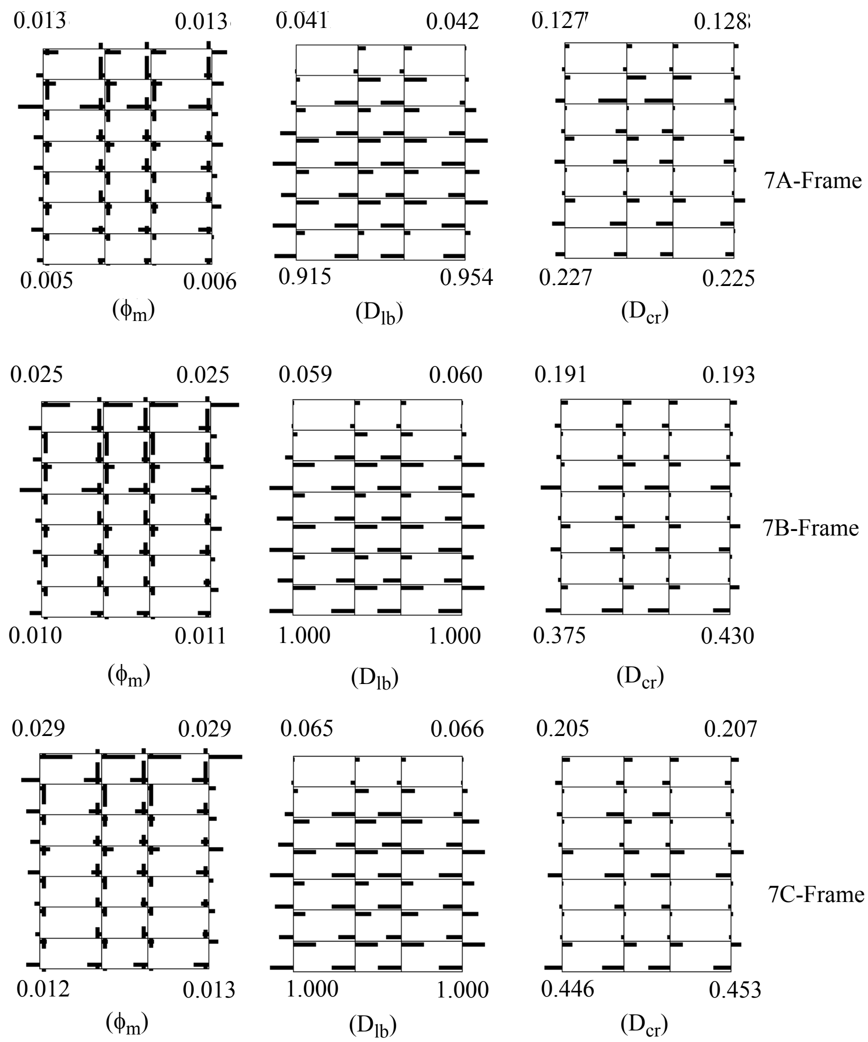


Fig.10 Damages of 7-story CFT frame under strong ground motion (JMA-KOBE NS & UD)
($\Delta C=0.3$, $\rho=3.0$ (D/t=93), $r_c=1.5$, $\sigma_c=60\text{N/mm}^2$)

5.2 Effect of shear-strength distribution on $(D_{cr})_m$

Concerning with the distribution of story-shear strength ratio, there are three kinds of CFT frame expressed by A-Frame, B-Frame and C-Frame. The seismic response calculations of these frames under strong ground motions of JMA-KOBE NS & UD and ART NS & UD are carried out. The obtained damage and damage ratio distributions of the three kinds of CFT frame are shown in Fig.11. It is shown that the concentration of crack damage ratio is effected only by the deviation of story-shear strength ratio (ΔC) from the basic distribution (C_{oi}) and it is not effected significantly by the distribution type itself of the story-shear strength ratio. According to the relations between the value of $(D_{cr})_m$ and the deviation of story-shear strength (ΔC) in Fig.11, we can see that the maximum crack damage ratio $(D_{cr})_m$ increases monotonically with the deviation of story-shear strength ratio (ΔC) even if the other design conditions are different.

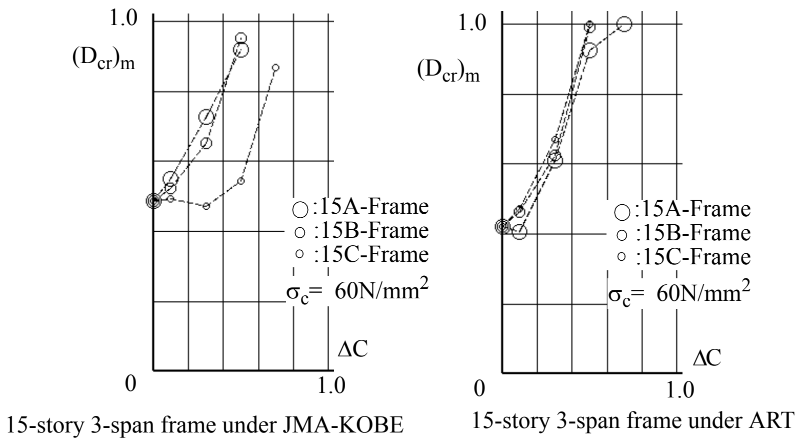


Fig.11 Effect of ΔC on the maximum crack damage factor $(D_{cr})_m$ of 15-story CFT frame (ρ=3.0 (D/t=93), r_c=1.5, σ_c=60N/mm^2)

5.3 Effect of column-over-design factor on $(D_{cr})_m$

The effect of the bending strength ratio of CFT column to H-section beam (r_c) on the seismic response and the damage of CFT frame under strong ground motion is examined here. The relations between the column-over-design factor and the maximum damage ratios of CFT frame are explained by some calculated results in Fig.12. The figure shows the maximum elastic-plastic hinge rotation $(\phi/\phi_u)_m$, the maximum local buckling damage ratio $(D_{lb})_m$ and the maximum crack damage ratio $(D_{cr})_m$ of CFT column. According to Fig.12, it is shown that the maximum damage and damage ratio are clearly effected by the column-over-design factor (r_c) and all of them decrease monotonically with it.

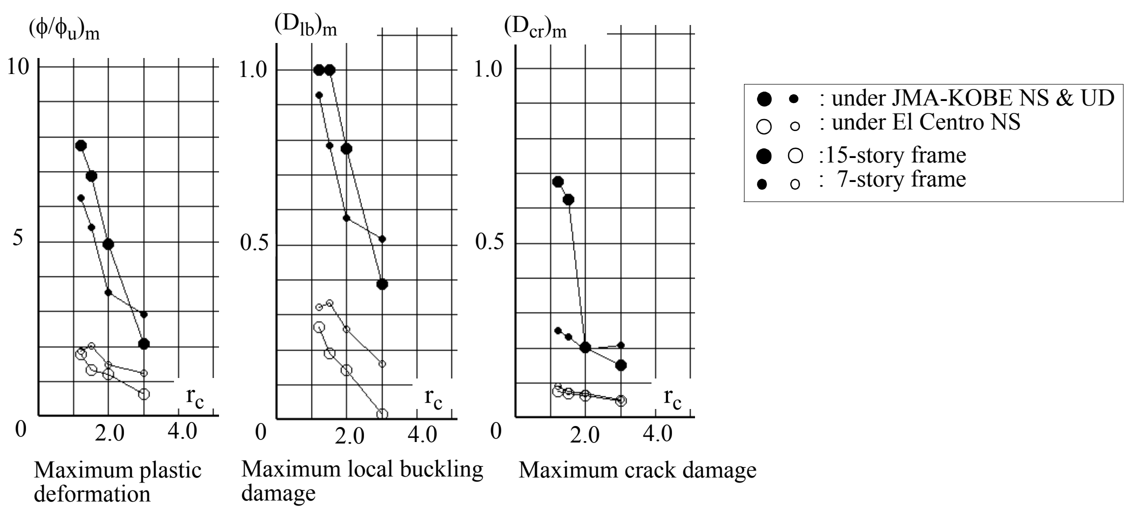


Fig.12 Effect of r_c on the maximum damages of 7-story and 15-story CFT frames (ρ=3.0 (D/t=93), σ_c=60N/mm^2, ΔC=0)

5.4 Effect of concrete-to-steel tube strength ratio on $(D_{cr})_m$

The strength ratio of filled concrete to steel tube of CFT column (ρ) effects on the crack and crack development of steel tube under strong seismic load. Seismic responses of CFT frame with variable ρ-values are calculated and the maximum response results of the elastic-plastic hinge rotation of CFT column, the local buckling damage ratio and the crack damage ratio are obtained. To show the effect of ρ-values on the maximum damage and maximum damage ratios, some of the calculated results are shown in Fig.13 as examples.

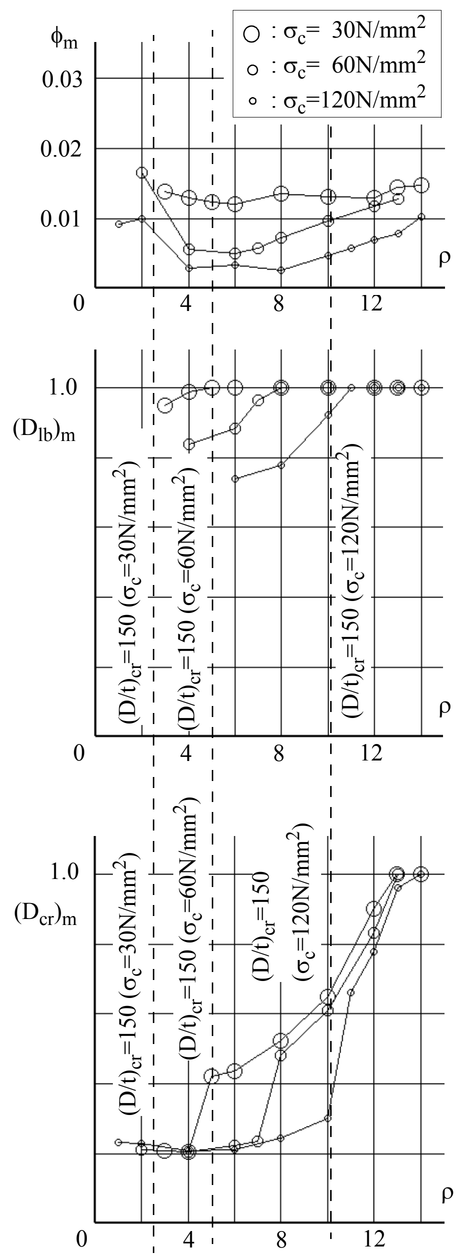


Fig.13 Effect of ρ on the maximum damages of 15-story CFT frames under JMA-KOBE NS & UD (r_c=1.2, ΔC=0)

According to the figures, the complicated differences due to the design conditions are observed but the damage ratios of local buckling and steel tube crack in all cases increase monotonically with the concrete-to-tube strength ratio of CFT column (ρ). It is also observed that the CFT frame whose concrete-to-tube strength ratio of CFT column (ρ) is in the range of $\rho<11$ does not crack. From this result it is shown that there is the upper bound of the concrete-to-tube strength ratio (ρ) for CFT frame not to crack under strong seismic load.

5.5 Earthquake resistant parameter (ξ)

Many seismic response and dynamic collapse of CFT frame under strong ground motion have been calculated and it is shown that the dynamic collapse of CFT frame is closely related to the first crack of CFT column. The maximum crack damage ratio $(D_{cr})_m$ which corresponds to the first crack of CFT column in the frame is mainly effected by the shear-strength distribution (ΔC), the column-over-design factor (r_c) and the strength ratio of filled concrete to steel tube of CFT column (ρ). Every factor (ΔC, r_c, ρ) effects on the maximum crack damage ratio of CFT column independently. Every factor also changes the maximum crack damage ratio monotonically. Further more every factor has the criteria for CFT frame not to crack even under strong ground motion like JMA-KOBE NS & UD or ART NS & UD. The criteria of the factors have been obtained as $r_c=2.0$, $\rho=11.0$ and $\Delta C=1$.

From these results we can express the effect of the factors on the ratio of safety $(1-(D_{cr})_m)$ by $(r_c/2.0)^{C1}$, $(1-\rho/11.0)^{C2}$ and $(1-\Delta C)^{C3}$, (C_1, C_2, C_3: constants). Because all these factors work on the safety ratio of any CFT frame under strong ground motion, the effects of these factors on it are expressed by the product of them and given by Eq.(5).

$$\xi = \left(\frac{r_c}{2.0}\right)^{C_1}\left(1-\frac{\rho}{11.0}\right)^{C_2}(1-\Delta C)^{C_3} \qquad (5)$$

The value of ξ in this equation is also corresponding to the parameter to express the ultimate earthquake resistant capacity of CFT frame.

5.6 Formula of earthquake resistant capacity

As mentioned above, the safety ratio of CFT frame is expressed by the use of the parameter ξ.

$$1-(D_{cr})_m=C_0\,\xi \qquad (6)$$

in which C_0 is constant and not related to ξ.

From Eq.(6) the maximum damage ratio of steel tube crack $(D_{cr})_m$ is obtained.

$$(D_{cr})_m=1-C_0\,\xi \qquad (7)$$

This equation shows not only the maximum damage ratio of CFT column but also the damage ratio of CFT frame under strong seismic load like JMA-KOBE NS & UD.

On the basis of the calculated damage ratios in the seismic response analysis, the constants C_1, C_2, C_3 are decided by the use of the least-mean-square method. The obtained values are in Table-2.

Table-2 Parameters (C_0, C_1, C_2, C_3) in Eq.(5)-Eq.(7)

CFT Frame	σ_c(N/mm^2)	C_0	C_1	C_2	C_3
7-story 3-bay frame	30	1.13	1.97	2.91	1.88
	60		1.64	0.87	1.69
	120		0.79	0.55	0.87
15-story 3-bay frame	60	1.19	1.93	0.55	2.32

Seismic responses of many CFT frames designed under variable conditions are numerically analyzed by the proposed analysis method and compared with the proposed ξ-$(D_{cr})_m$ relation in Fig.14-Fig.15. The damage ratios calculated by the numerical analysis method are shown by the dots. In the figures we can see the damage ratios of the calculated seismic response distribute linearly and they are well predicted by the proposed ξ-$(D_{cr})_m$ relation in every case. From these results it is ascertained that the obtained equation is useful to predict the ultimate earthquake resistant capacity defined by the dynamic collapse of CFT frame under strong seismic load like JMA-KOBE NS & UD and ART NS & UD.

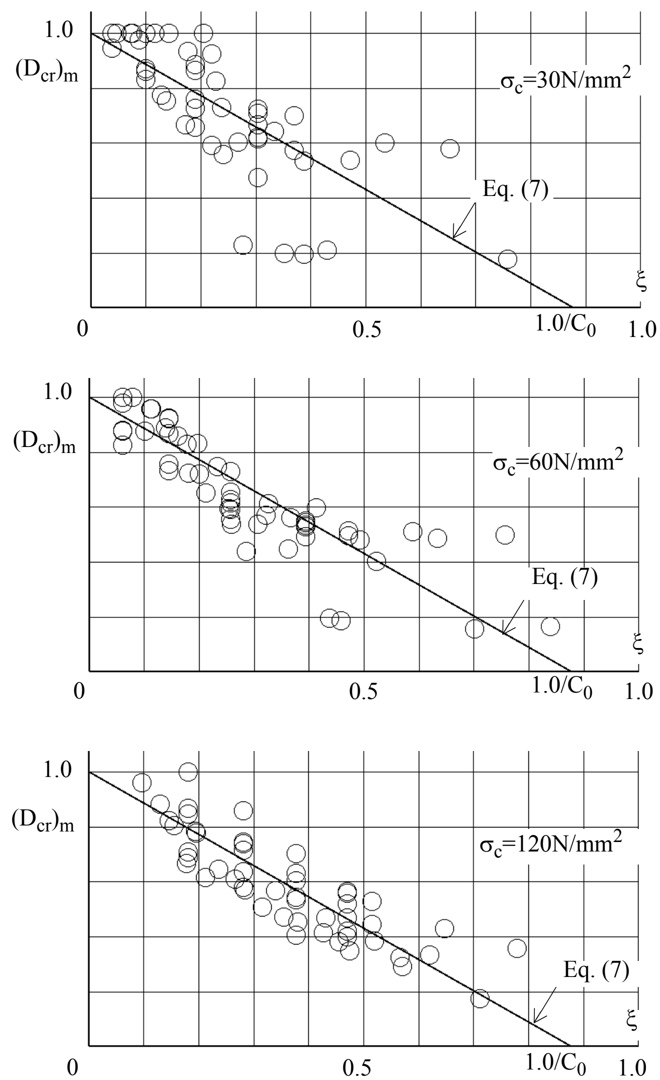


Fig.14 Relation between ξ and damage ratio of 7A-Frame,7B-Frame,7C-Frame under JMA-KOBE NS & UD and ART NS & UD

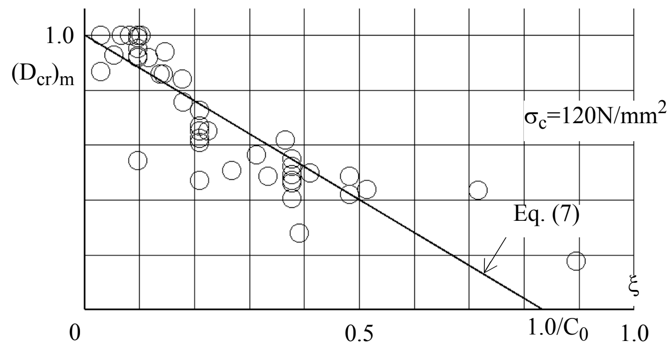


Fig.15 Relation between ξ and damage ratio of 15A-Frame,15B-Frame,15C-Frame under JMA-KOBE NS & UD and ART NS & UD

6. CONCLUSIONS

The seismic response and damage of many CFT frames under strong ground motion have been analyzed and it is pointed out that the ultimate earthquake resistant capacity of CFT frame, which is defined on the basis of the dynamic collapse of frame, is strongly effected by the story-shear strength distribution, the column-over-design factor and the filled concrete-to-steel tube strength ratio.

To predict the ultimate earthquake resistant capacity of CFT frame and the effects of these design factors on it, a simple formula (Eq.(7)) is proposed. It is ascertained that the formula is simple but it can approximate well the damage ratio of CFT frame which expresses the ultimate earthquake resistant capacity of it.

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