ASEISMIC CAPACITY OF REINFORCED CONCRETE STRUCTURES

Hiroshi KAWAMURA

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INTRODUCTION

Japan is a well known seismic country.

At the rate of about one time in two years, destructive earthquakes with more than 7.0 magnitude have attacked the Japanese Islands and the regions adjacent to them these last decades. Since the Kwanto Earthquake (Magnitude 7.9), by which many buildings in the capital city of Japan, Tokyo, were destroyed in 1923, aseismic design and construction of buildings have been made obligatory to the all structural engineers.

Earthquake structural engineering with about a half century history is now in progress through many researches on earthquakes, earthquake structural damages and structural aseismic characteristics.

1-1. Outlines of Recent Architectural Buildings and Present Aseismic Design

A great majority of the numbers of buildings is occupied by wooden houses, which are forbidden to have more than two stories. A large part of public buildings such as schools, hospitals, offices and so on is constructed of reinforced concrete structures with rigid frames or with shear walls. Such public buildings with more than 6~7 stories are constructed of steel reinforced concrete structures or steel structures. The height of these buildings had been forbidden to exceed 31 meters before 1963, intending for rigid type structures with short natural periods.

According to the progresses of modern city functions and aseismic design and construction techniques of high rise buildings, the limitation of height in Japanese Building Law was abolished in 1963. Then, recently, many high rise buildings with more than 40 stories or 100 meter height have been constructed as flexible and soft type steel structures with long natural periods. As a special case, even a reinforced concrete building with 18 stories has been constructed.

Usual type buildings with less than 31 meter height are designed against the lateral loading of more than 0.2 times dead and live loads. When every working stresses of structures subjected to such lateral loading are under specified temporary allowable stresses, the buildings are considered to have enough aseismic capacity. As for wooden houses or wall-type reinforced concrete buildings, the necessary minimum quantities of wooden bracings or reinforced concrete shear walls are given, so exact structural analyses are not needed.

Regarding high rise multi-story buildings, dynamic response analysis is usually applied to aseismic design. Using ground acceleration waves, vibrational models and restoring force functions of buildings, relative story response deformations given by means of numerical integration of motion-equations are adopted as an aseismic safety criterion.

1-2. Recent Earthquake Damages of Buildings

The most remarkable features of recent earthquake damages of buildings are the fractures of reinforced concrete buildings which have been designed in accordance with modern aseismic codes. Fortunately, steel reinforced concrete buildings and high rise steel buildings have not yet experienced destructive earthquakes. So, in reality, their aseismic capacity and safety have not yet verified. The causes of earthquake damages of wooden houses are their old ages, landslips, tsunamis and so on.

The typical fracture modes of reinforced concrete buildings were as follows (1)(2)(3):

(A) Shear fractures of short columns through spandrels of reinforced concrete schools, and excessive torsional horizontal deformations of reinforced concrete buildings with asymmetrically arranged shear walls in plan at the Tokachi-Oki Earthquake (M. 7.9), 1968, Japan.

(B) Excessive lateral deflection of a modern reinforced concrete hospital at the San Fernando Earthquake (M. 6.6), 1971, USA.

(C) Crushing down of the base story of a reinforced concrete hotel with few shear walls at the Ooita Earthquake (M. 6.4), 1975, Japan.

A fortunate and remarkable fact ,however, is that the reinforced concrete buildings with sufficient shear walls have not suffered any damages even by destructive earthquakes.

1-3. Recent Researches on Aseismic Characteristics of Reinforced Concrete Structures

Earthquake structural Engineering is considered to consist of the following main three kinds of field;

(A) Earthquake Excitations,

(B) Structural Response Characteristics.

(C) Aseismic Characteristics of Structures, During earthquakes, buildings are subjected to earthquake excitations and show responses in accordance with aseismic characteristics of them. Finally, the degree of their damages and failures must be estimated on the basis of their aseismic characteristics again. Consequently, these three fields should be investigated and combined on a common basis to establish a reasonable aseismic design method. From the technological point of view, the knowledges about aseismic behaviors of structures give the most important bases of the researches on earthquake structural engineering. Without these knowledges, the all investigations and discussions on the aseismic safety and capacity of buildings produce only abstract infomations. In this chapter, the outlines of researches on the aseismic behaviors of reinforced concrete structures are mainly explained.

In advance to the researches on aseismic properties of structures, there must be some assumptions of earthquake excitations and structural responses. In Japan, according to the following two main aseismic methods, that is, static seismic coefficient (0.2) method and dynamic response analysis method, one way load-deformation behaviors and restoring force functions under cyclic loading of structural members and structures have been investigated. Especially, one way properties until fractures are expected to be applicable to the ultimate strength design method. Recently, some other type aseismic design method, that is, ultimate aseismic design methods based on structural fractures are proposed. Low cycle fatigue, energy dissipation capacity and fracture criterion, therefore, are investigated as new aseismic factors of structures.

The outlines of researches on the aseismic behaviors of reinforced concrete structural members and structures are described as follows.

Monotonic and Cyclic Behaviors Under Static Loads

Usual type- reinforced concrete buildings are composed of rigid frames with the following main three kinds of aseismic elements, that is, long columns, short columns and shear walls. In Japan, long columns are designed for predominant bending moments and show flexural yielding without buckling. Short columns and shear walls show predominant shear deformations and shear fracture modes.

Monotonic elasto-plastic flexural mechanics and load-deformation behaviors of long columns were already clarified experimentally and analytically (4). Cyclic behaviors of them were also made clear from the point of flexural mechanics, but many facts are yet left unknown in regard to cyclic behaviors until fracture (5)(6)(7). Low cycle fatigue and fracture criteria of them are partially clarified (8). As for beams, cyclic tests and analyses had been already carried out by foreign researchers (9). Very few systematic investigations, however, have been performed especially taking into account the effects of axial loads in foreign countries (10).

It had been pointed out by Dr. Yamada before the Tokachioki Earthquake that when a long column becomes shorter due to spandrels, its flexural yielding mode is suddenly displaced by shear fracture mode (11). These warning data were verified by the damages of reinforced concrete buildings at that Tokachioki Earthquake (12). Part of the effects of shear span ratios, shear reinforcements and axial load levels on the monotonic behaviors of short columns is already clarified. As for cyclic behaviors of them, experimental data are now yet under the state of collection. Low cycle fatigue and fracture criteria of them are partially made clear experimentally (13). Generally speaking, there still remain many obscure parts about fundamental mechanics of shear failure of reinforced concrete beams and columns (14).

The aseismic effectiveness of shear walls had been found and researched by Naito before the Kwanto Earthquake, and this was verified by that earthquake (15). The elastic behaviors of them, therefore, have been already investigated since the early times. Recently, there are many reinforced concrete buildings with a few or no shear walls due to planning functions, and the damages of such buildings have been always severe when subjected to destructive earthquake loadings. Even the researches on monotonic properties until fracture of shear walls have been begun with in these years (16), and very few are yet known on cyclic behaviors (17).

As other aseismic problems of reinforced concrete structures, there are the behaviors of beams, beams with slabs and connections between beams and columns and of bond mechanics between concrete and reinforcing bars. It was shown that only beams with rectangular cross sections have enough deformation ductility, but there remain many unknown factors in regard to beams with slabs. The slipping mechanics of cyclic bond behaviors are under researches (18).

A true aseismic capacity of a reinforced concrete building as an entire structure composed of structural elements should be evaluated by combining the aseismic properties of these aseismic elements, that is, long columns, short columns, shear walls and so on. Researches from this point, however, are very few.

Cyclic Behaviors under Dynamic Loads

In accordance with the progresses of electronic digital computer, the numerical integration of equations of motion became easy, which have enabled us to design high rise buildings.

In order to apply this dynamic response analysis method to reinforced concrete structures, their restoring force characteristics must be made clear. From this point of view, dynamic tests and analyses on reinforced concrete columns and rigid frames have been tried (19)(20). Although well coincidences between experimental and computed results are verified, these researches are not carried out untill ultimate states of structures, i.e., fracture phenomena.

1-4. Ultimate Aseismic Design of Structures

Ultimate aseismic design of structures is defined to consist in controlling the ultimate states of structures, i.e. fractures. As physical factors by which the ultimate states are controlled, the followings are able to be considered; strength capacity, energy absorption capacity and numbers of cyclic loading until fracture.

As for strength capacity, "Seismic Coefficient Method" proposed by Dr. Sano in 1916 is the origin. As for energy absorption capacity, Dr. Tanabashi proposed "Velocity-Potential Energy Method" in 1935. The former is effective to rigid type structures with short natural periods and poor ductility, and the latter to soft type structures with long natural periods and sufficient ductility. Furthermore, if earthquake excitations are able to be considered as one pulse these two methods are very reasonable.

In reality, however, earthquake excitations continues with some durations and nearly constant intensity amplitudes. Consequently, fatigue characteristics of structures have to be taken into account.

"Resonance-Fatigue Method" proposed by Dr. Yamada and the author was introduced to satisfy this demand. In "Resonance-Fatigue Method", a new aseismic concept, "RESONANCE CAPACITY" is adopted as a controlling fatigue factor and as an aseismic criterion of structures under the idealized ultimate vibrational state, i.e. steady-state resonance. Theoretical and experimental bases are explained in Chapter 3.

Recently, "Dynamic Response Analysis" is considered to be the newest, the most general and of great promise (21)(22). This method makes clear the deformation processes of structures in elastic and plastic ranges, but this method never belongs to the category of ultimate aseismic design.

1-5. The Purpose and Composition of This Thesis

The fact that reinforced concrete modern buildings always collapse during destructive earthquakes seems to be an evidence of our lack of cognizance of the fracture phenomena of reinforced concrete structures. There exist and will be constructed many and many reinforced concrete structures in many countries as well as Japan, so it is the most important and emergent subject for us to clarify aseismic behaviors and to establish a reasonable ultimate aseismic design method of reinforced concrete structures.

Consequently, the purpose of this paper is to investigate the ultimate aseismic capacity of reinforced concrete structures based upon the new aseismic concept, "Resonance Capacity" criterion.

In order to discuss and evaluate the ultimate aseismic capacity, this thesis is composed as follows:

In Chapter 2, medium or low rise reinforced concrete structures composed of three kinds of aseismic elements, i.e. long columns, short columns and shear walls are divided into two aseismic types, rigid and flexible ones based upon load-deformation relations. In Chapter 3, a new fundamental concept "Resonance Capacity" criterion is proposed to evaluate the aseismic capacity of reinforced concrete structures on the basis of the hysteretic area of load-deformation loops and the fatigue characteristics of the three-type aseismic elements. In Chapter 4, based upon the new fundamental concept introduced in Chapter 3, the aseismic capacity and safety are investigated regarding two types of medium or low rise reinforced concrete structures, i.e. rigid ones with shear walls and flexible ones without shear walls. In Chapters 5, 6, the ultimate aseismic capacities of low-rise reinforced concrete buildings with asymmetric shear walls and multi-story buildings with and without cantilever-type shear walls are evaluated, respectively.

This thesis is arranged and composed on the basis of the papers reported by the author which were written under the guidances of Prof. Dr. Yamada as follows:

Chapter 2 ; from Refs. (23)(24),

Chapter 3 ; from Refs. (23)(25),

Chapter 4 ; from Refs. (24)(26),

Chapter 5 ; from Ref. (27),

Chapter 6 ; from Refs. (28)(29).

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CHAPTER 2

ASEISMIC CLASSIFICATION OF REINFORCED CONCRETE STRUCTURES

2-1. Introduction

Before the evaluation of aseismic capacity of reinforced concrete structures, the outlines of aseismic characteristics of them should be made clear. In order to comprehend such aseismic behaviors, reinforced concrete structures are idealized into several categories according to their behaviors as observed in real earthquakes and laboratory experiments. It is proposed that reinforced concrete structures are composed of three aseismic elements, i.e. long columns, short columns and shear walls, the former shows flexural yielding type and the latter two show shear fracture type.
In accordance with the various combinations of these three aseismic elements, each of which shows different load-deformation relationships, the general characterization of reinforced concrete structures becomes possible from the aseismic point of view.

2-2. Classification of Reinforced Concrete Aseismic Elements

Fracture Modes of Reinforced Concrete Members

At the Tokachi-Oki-Earthquake, May 16, 1968, Japan, the shear explosion failure of restrained short columns with small shear span ratios was the most remarkable damage characteristics of reinforced concrete school buildings (1)(2)(3)(4). At the San Fernando-Earthquake in California, Feb. 9, 1971, U.S.A., the extreme deflections of reinforced concrete long columns caused the heavy damage of the Olive View Hospital (5). On the other hand, it is observed at any earthquakes that symmetrically arranged reinforced concrete shear walls with sufficient cross sectional area play frequently the most effective role as stiffeners of buildings. These three typical phenomena imply the essential and significant factors to characterize the behaviors of reinforced concrete structures subjected to earthquake excitations. Each of the three types of structural members, short columns, long columns and shear walls, must have its own aseismic characteristics.

The fracture modes of reinforced concrete structural members are able to be classified mainly into two types, i.e., shear failure and flexural yielding (6). The former is often observed in the case of panel members subjected to predominant shear force, and the latter in the case of linear members subjected to predominant bending moment. When the reinforced concrete structures composed of such linear element as columns and beams, and of panel elements such as slabs and shear walls, undergo lateral forces such as earthquake excitation, short columns and shear walls show shear failure mode and long columns show flexural yielding. Here, in this study, the failure of beams is neglected, because of their mechanical improvements due to beam-slab interactions.

Typical fracture modes such as mentioned above are shown in Fig. 1 (1)(7)(8)(9). The names of test specimens in this figure, SW, SC and LC, represent shear wall, short column and long column, respectively. The ordinate, V, and the abscissa, R, represent story shear force and relative story displacement angle, and the triangles, TC, SC and CC, indicate the formation of tensile, shear and compressive cracks, respectively. Fig. 1 shows that shear failure mode is characterized by the diagonal cracks and shear compression collapse of diagonal concrete elements of SW and SC specimens, and that flexural yielding type is characterized by the tensile cracks at the tension side and the compressive cracks at the compression side of LC specimen. Furthermore, it is a remarkable feature that SW and SC are so brittle that the more deformation capacity than 0,004 - 0,005 rad. is unable to be assumed, while LC has sufficient ductility.

Three Kinds of Aseismic Elements: Shear Wall, Short and Long Column

According to the fracture modes shown in Fig. 1, reinforced concrete structural members are able to be classified mainly into three aseismic elements, i.e., SW, SC and LC. Reinforced concrete structures, therefore, are considered to be composed of these three kinds of aseismic elements. These aseismic elements and their deformation modes are shown schematically in Fig. 2, where SC element is given as LC with walls in the lower part of story height.

Assumptions for the Deformation of Aseismic Elements

If the lateral load - displacement relationships of aseismic elements, LC, SC and SW, are given, the total load - displacement relationship of reinforced concrete structure composed of these aseismic elements is able to be computed, and the classification of reinforced concrete structures becomes possible on the basis of their aseismic characteristics (10). In order to calculate the simplified load - displacement relationships of aseismic elements, the followings are assumed:

Assumptions for the deformation of LC and SC elements

The lateral story displacement of LC and SC elements, which are restrained against rotations at the both ends and subjected to story shear force V, is given by the superposition of flexural and shear deformations, each of which is calculated independently on the basis of the following assumptions (See Fig. 3) (11).

(A) As for the flexural deformation of LC and SC elements --

(1) The normal stress - strain relationship of concrete confined with ties and of reinforcing steels are perfectly elasto-plastic (See Fig. 4) (7).

(2) Cross section of reinforced concrete columns is idealized to be composed of equivalent five concentrated mass points, i.e., three points of concrete at compressive, centroidal and tensile positions, and two points of compressive and tensile reinforcing steels (See Fig. 5) (11)(12)(13)(14).

(3) Stresses, strains and external forces, M and N of the idealized reinforced concrete cross section in the case of flexural yielding are distributed such as shown in Fig. 6.

(4) Moment - curvature relationship of the idealized cross section is assumed to be perfectly elasto-plastic (See Fig. 7).

(5) The effects of shear and $P\Delta$ effect are neglected.

(B) <u>As for the shear deformation of LC and SC</u> --

(1) The shear stress - strain relationship of concrete is perfectly elasto-plastic when confined with sufficient shear reinforcement ($\rho_h >$ 1%), and has no ductility when confined with poor shear reinforcement ($\rho_h \fallingdotseq 0\%$) (See Fig. 8-a) (1)(3)(15)(16).

(2) The fracture criterion of concrete under normal and shear stresses is assumed to be elliptic (17) such as shown in Fig. 8-b, and expressed by:

$$\frac{\tau_y}{f'_c} = \sqrt{-0.10\left(\frac{\sigma}{f'_c}\right)^2 + 0.09\left(\frac{\sigma}{f'_c}\right) + 0.01} = F\left(\frac{\sigma}{f'_c}\right) . \qquad (1)$$

(3) The area of shear-resisting cross section is assumed to be:

$$A_{cs} = \frac{7}{8}\,bd, \qquad (2)$$

and the shear failure of reinforced concrete elements is caused by the collapse of the concrete at the midspan of story height.

(4) The effects of bending moment and the dowel action of longitudinal reinforcements are neglected.

Assumptions for the shear deformation of SW elements

(1) If reinforced concrete shear walls are confined with sufficiently rigid beams and columns, they are able to be replaced by equivalent compression braces of concrete (9). Here, SW element is replaced by an equivalent compressive concrete brace with effective width B_e such as shown in Fig. 9.

(2) The normal stress - strain relationship of the concrete brace has no ductility such as shown in Fig. 10 (9).

Load - Displacement Relationships of Aseismic Elements

On the basis of the assumptions described above, the story shear force V and the relative story displacement δ relationships of aseismic elements, LC, SC and SW, are shown in Fig. 11, and their critical values of V and δ are expressed by the following equations:

for LC element in which $\bar{V}_y^B \leqq \bar{V}_y^S$;

$$V_{LCy} = \bar{V}_y^B f'_c bh, \qquad \delta_{LC} = (R_y^B + R_y^S \frac{\bar{V}_y^B}{\bar{V}_y^S}) H, \tag{3}$$

for SC element in which $\bar{V}_y^B \geqq \bar{V}_y^S$;

$$V_{SCy} = \bar{V}_y^S f'_c bh, \qquad \delta_{SC} = (R_y^B \frac{\bar{V}_y^S}{\bar{V}_y^B} + R_y^S) H' , \tag{4}$$

for SW element;

$$V_{SWy} = (\varepsilon \frac{2}{3} \sin\theta \ \cos\theta) f'_c Lt, \qquad \delta_{SW} = \frac{{}_c\varepsilon_m}{\sin\theta \cos\theta} H. \tag{5}$$

The characteristic values, $\bar{V}^B_y$, R^B_y, $\bar{V}^S_y$ and R^S_y are given as follows:

$$\bar{V}^B_y = \frac{V^B_y}{f'_c bh} = \frac{1}{\bar{H}} \{X + 2(1 + X)\omega\}\gamma , \tag{6}$$

$$\text{where} \qquad 0 < X < \frac{0.3 + \omega(\frac{1-\chi}{\chi})}{1 + 2\omega} , \tag{7}$$

$$R^B_y = \frac{\bar{V}^B_y \bar{H}^2}{12\ \overline{EI}} , \tag{8}$$

$$\text{where} \qquad \overline{EI} = \frac{\frac{1}{2} \{X + 2(1 + X)\omega\} (\gamma)^2 \ (0.3\chi + \omega)}{\chi_c \varepsilon_y \{0.3\chi + X + 2(1 + X)\omega\}} , \tag{9}$$

$$\bar{V}^S_y = \frac{V^S_y}{f'_c bh} = \frac{A_{cs}}{bh} F(\frac{\sigma}{f'_c}) = \frac{A_{cs}}{bh} F(X)$$

$$= \frac{7}{8} \frac{d}{h} \sqrt{-0.10(X)^2 + 0.09(X) + 0.01} , \tag{10}$$

$$R^S_y = \frac{f'_c}{G_c} F(X) . \tag{11}$$

The Critical Shear - Story Height - Ratio

As the critical value of story height of columns, by which the reinforced concrete columns are classified into two types, i.e., SC and LC, the "Critical Shear Story Height Ratio $\bar{H}_{cr}$" is derived from the condition that

$$\bar{V}^B_y = \bar{V}^S_y , \tag{12}$$

as follows (3)(11):

$$\bar{H}_{cr} = \frac{\{X + 2(1 + X)\omega\}\gamma}{(\frac{7}{8})\ \frac{d}{h}\ F(X)}. \tag{13}$$

Consequently, the clear spans of story height of LC and SC should satisfy the following condition:

$$\text{for LC,}\quad \bar{H} \geqq \bar{H}_{cr}, \qquad \text{for SC,}\quad \bar{H}' \leqq \bar{H}_{cr}. \tag{14}$$

Zoning of SC and LC columns due to Eq. 13 is shown in Fig. 12.

2-3. Classification of Reinforced Concrete Structures

Total Load - Displacement Relationships of Reinforced Concrete Structures

With the consideration of the deformation of beams adjacent to LC element, here in this paper, instead of δ_{LC}, $2\delta_{LC}$ is assumed as the flexural yielding deflection of LC element. The beams adjacent to SC element are usually so deep and rigid that their deformation may be negligible. As for the usual types of medium or low rise reinforced concrete structures, there is the following relation among the critical relative story displacements of aseismic elements:

$$\delta_{SC} < \delta_{SW} < \delta_{LC}\ . \tag{15}$$

Furthermore, in reinforced concrete structures with symmetrically arranged shear walls in plan, the relative story displacement of all elements have the same value through the rigid panel action of floor slabs. Therefore, when a reinforced concrete structure is composed of α pieces of LC element, β pieces of SC element and γ pieces of SW element, its total V - δ relationship is able to be illustrated such as shown in Fig. 13,

where

$$
\begin{aligned}
V_{LC} &= \alpha V_{LCy} + (\beta V_{SCy}) , \\
V_{SC} &= \alpha V_{LCy}\frac{\delta_{SC}}{\delta_{LC}} + \beta V_{SCy} + \gamma V_{SWy}\frac{\delta_{SC}}{\delta_{SW}} , \\
V_{SW} &= \alpha V_{LCy}\frac{\delta_{SW}}{\delta_{LC}} + (\beta V_{SCy}) + \gamma V_{SWy} ,
\end{aligned} \tag{16}
$$

in which the value (βV_{SCy}) is able to be considered when SC elements have sufficient shear reinforcement ($\rho h \geqq 1\%$) and the dotted lines in the Fig. 13 correspond to such a case.

Fracture Modes of Reinforced Concrete Structures

According to the critical displacement corresponding to the maximum V-value, reinforced concrete structures are classified into three types such as shown in Fig. 14, where these types are nominated as follows:

$$
\begin{aligned}
&\text{LC Fracture Mode, when } V_{LC} > V_{SL}, V_{SW}, \\
&\text{SC Fracture Mode, when } V_{SC} > V_{LC}, V_{SW}, \\
&\text{SW Fracture Mode, when } V_{SW} > V_{LC}, V_{SC}.
\end{aligned} \tag{17}
$$

These fracture modes are able to be zoned in the $\alpha/\gamma - \beta/\gamma$ plane such as shown in Fig. 15. Figs. 15(a) and (b) show the cases that SC elements have poor and sufficient shear reinforcement, respectively. The boundary lines A, B, C, between the fracture modes in Fig. 15 are easily given by the following equations:

$$V_{LC} = V_{SC} \quad \text{for A,}$$
$$V_{SC} = V_{SW} \quad \text{for B,} \qquad (18)$$
$$V_{SW} = V_{LC} \quad \text{for C.}$$

Rigid and Flexible Types of Reinforced Concrete Structures

Due to the facts that SC and SW elements have poor ductility and that LC elements have sufficient ductility, reinforced concrete structures are able to be classified into two types, i.e. rigid structures and flexible structures such as shown in Fig. 16. That is, SC and SW fracture modes belong to the former and LC fracture mode to the latter. This classification will have very important significances for the discussions on the aseismic capacity of reinforced concrete structures.

2-4. Concluding Remarks

Based upon the experimental results of the deformation and fracture behaviors of reinforced concrete structural members subjected to monotonic loadings, the classification of reinforced concrete members and structures are performed and their aseismic characteristics and capacity are discussed on. As a result, the followings are clarified:

(1) Reinforced concrete structural members are able to be classified

into three types of aseismic elements, i.e. Long Columns (LC), Short Columns (SC) and Shear Walls (SW). LC element shows the flexural yielding type and SC and SW elements show the shear fracture mode.

(2) Combining analytically the load - deflection relationships of the aseismic elements, reinforced concrete structures are able to be classified into three types which show LC, SC and SW fracture modes (See Fig. 15). On the basis of the aseismic characteristics, finally, reinforced concrete structures are able to be classified into two types, i.e. rigid structures and flexible structures (See Fig. 16).

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NOTATION

A_{cs} = shear resisting cross sectional area of columns

b = width of cross section of columns

d = distance from extreme compression fiber to centroid of tension reinforcement

d_s = distance from centroid of tension reinforcement to the tensile face of columns

E_c = modulus of elasticity of concrete

EI = flexural stiffeness of columns

$\overline{EI}$ = $EI/f'_c bh^3$

$F(\ )$ = function of fracture criterion of concrete under combined normal and shear stresses

f'_c = compressive strength of concrete

f_y = yield strength of reinforcement

G_c = modulus of shear of concrete ($= E_c/2(1 + \nu)$)

H = story height of long columns and shear walls

H' = story height of short columns

H_{cr} = critical story height (minimum one to show flexural yielding)

$\overline{H}$ = (H/h) story height ratio

$\overline{H}_{cr}$ = critical story height ratio

h = total depth of cross section of columns

L = span length

M = bending moment

N = axial force

N_o = ultimate strength of centrally loaded columns (= $(1 + 2\omega)\, f'_c bh$)

R = relative story displacement angle

R^B_y = flexural component of yielding R

R^S_y = shear component of yielding R

t = thickness of shear walls

V = story shear force

V_{LC} = V of structures when LC elements show flexural yielding

V_{SC} = V of structures when SC elements show shear yielding

V_{SW} = V of structures when SW elements show shear fracture

$\overline{V}_{LCy}$ = $V/f'_c bh$ of LC elements when they show flexural yielding

$\overline{V}_{SCy}$ = $V/f'_c bh$ of SC elements when they show shear yielding

$\overline{V}_{SWy}$ = $V/f'_c bh$ of SW elements when they show shear fracture

V^B_y = yielding V of columns when they show flexural yielding

$\overline{V}^B_y$ = $V^B_y/f'_c bh$

V^S_y = yielding V of columns when they show shear fracture

$\overline{V}^S_y$ = $V^S_y/f'_c bh$

X = axial load ratio, $(=N/N_o)$

SW = shear wall elements

SC = short column elements

LC = long column elements

α = number of pieces of LC elements

β = number of pieces of SC elements

γ = number of pieces of SW elements, shear strain, ratio of distance center-to-center of reinforcement on opposite faces of columns to h

δ = relative story displacement

δ_{LC} = δ at which LC elements show flexural yielding

δ_{SC} = δ at which SC elements show shear yielding

δ_{SW} = δ at which SW elements show shear fracture

${}_c\varepsilon_m$ = normal strain at the maximum stress of concrete

${}_c\varepsilon_y$ = idealized yield strain of concrete

${}_s\varepsilon_y$ = yield strain of reinforcing steel

θ = diagonal gradient angle of SW element , ν = Poisson's ratio

ρ = geometrical ratio of tensile reinforcement

ρ' = geometrical ratio of compressive reinforcement

ρ_h = shear reinforcement of columns and shear walls

σ = normal stress

τ = shear stress

τ_y = yield shear stress

Φ = curvature

Φ_y = yielding curvature

χ = ${}_s\varepsilon_y / {}_c\varepsilon_y$

ω = tensile reinforcement index (= $(fy/f_c')\rho$)

ω' = compressive reinforcement index (= $(f_y/f_c')\rho'$)

Supplements

RCW : specimen mark of shear wall (9)

RC:C1B : specimen mark of long column (7)

RC:C1Q : specimen mark of short column (1,8)

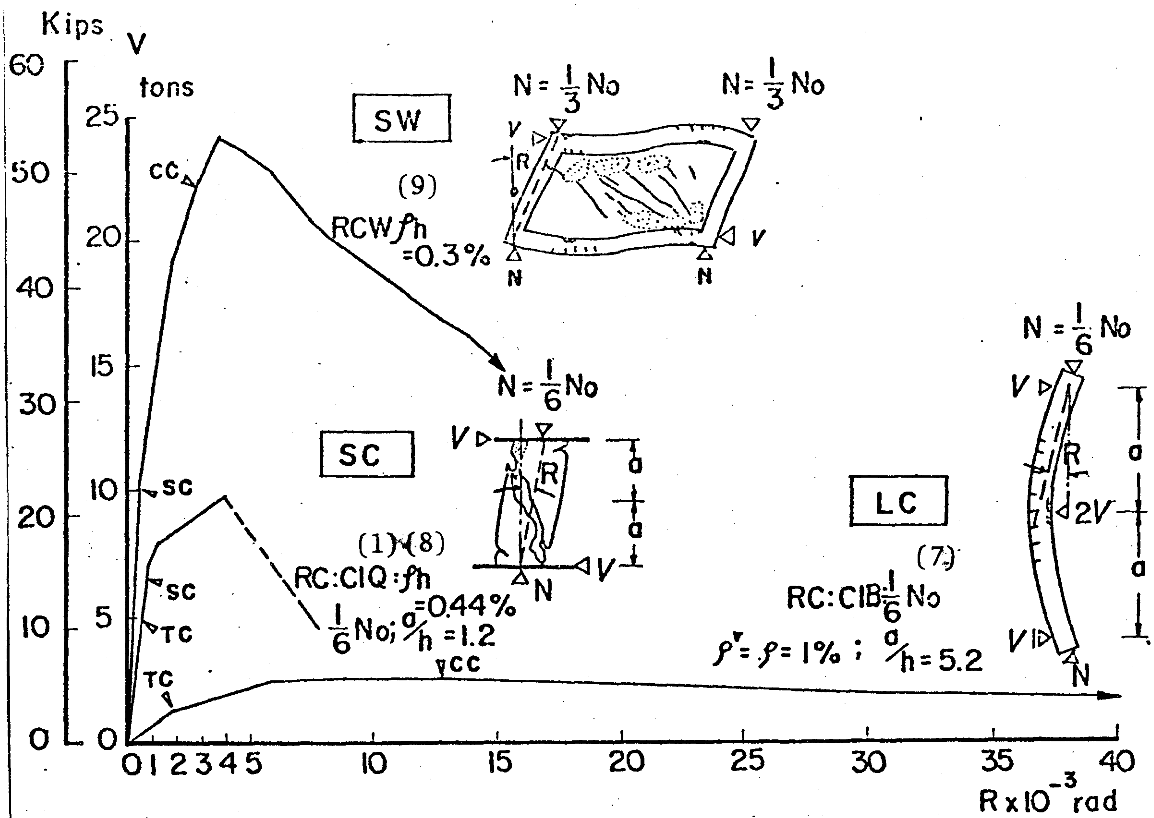


Fig. 1 Story Shear Force - Relative Story Displacement Relationships of Long Column (LC), Short Column (SC) and Shear Wall (SW)

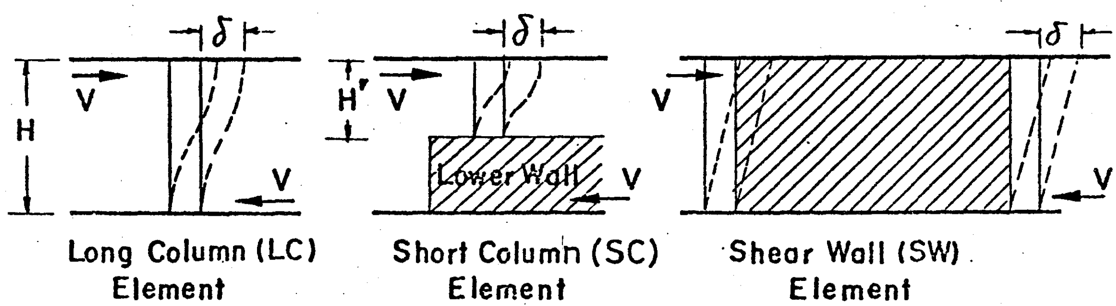


Fig. 2 Aseismic Elements

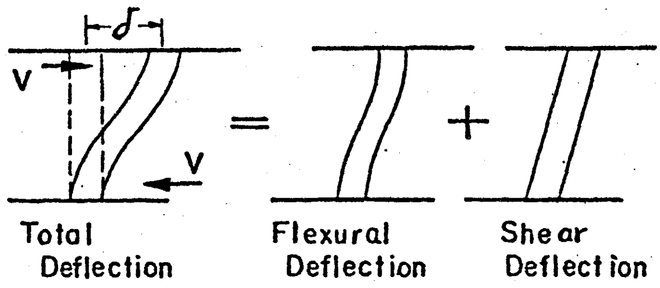
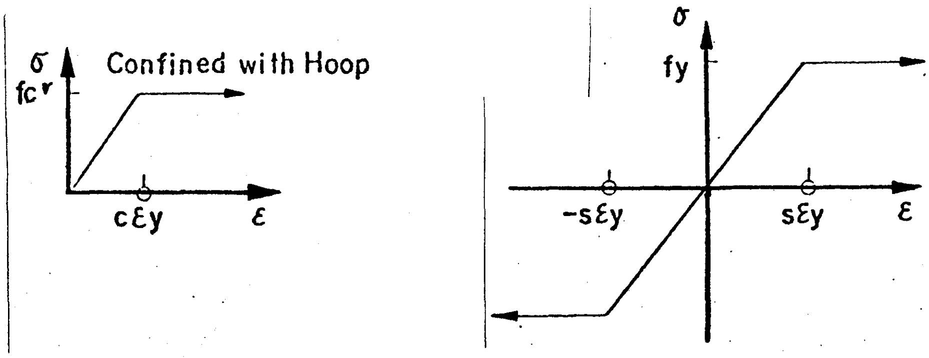


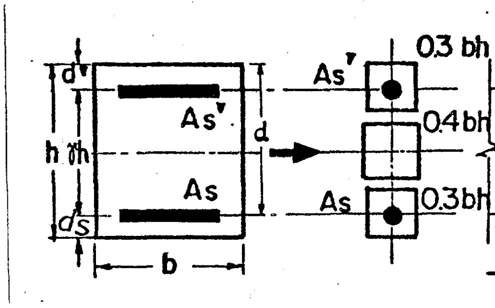
Fig. 3 Deflection of a Column



(a) Concrete

(b) Reinforcing Steel

Fig. 4 Stress-Strain Relationships of Materials



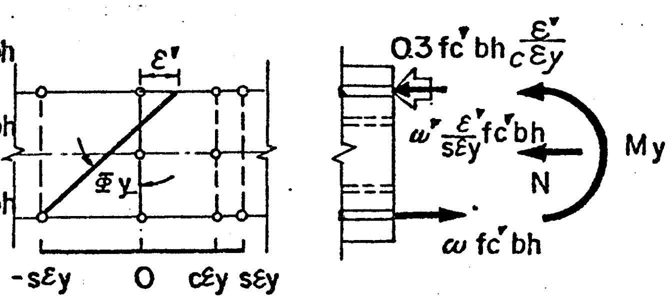


Fig.5 Idealization of a Reinforced Concrete Cross Section

Fig.6 Strain and Stress Distributions at Flexural Yielding Condition

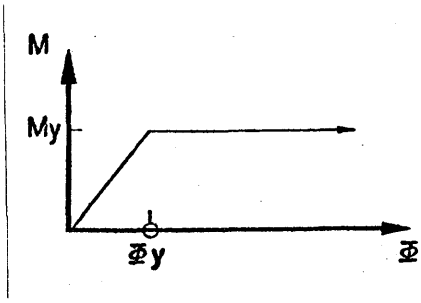


Fig. 7 Moment-Curvature Relationship of a Reinforced Concrete Cross Section

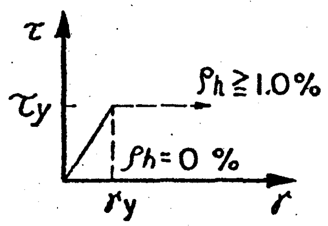


Fig.8(a)Shear Stress-Strain Relationship of Concrete

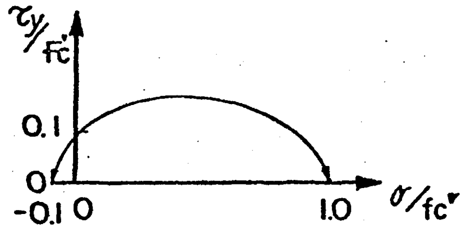


Fig.8(b)Fracture Criterion of Concrete Under Combined σ - τ Stresses

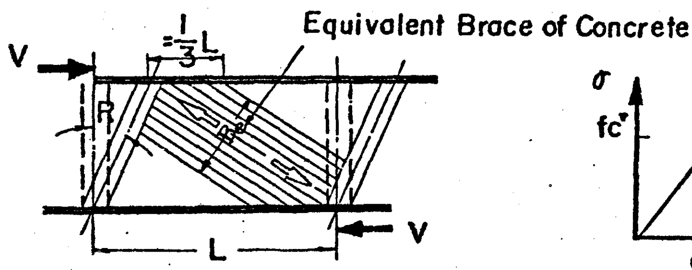


Fig. 9 Shear Deformation of Shear Wall

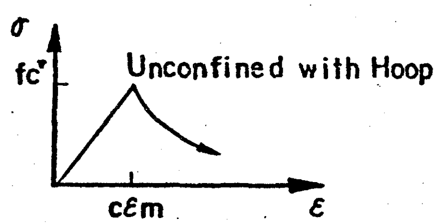


Fig. 10 Normal-Stress-Strain Relationship of Equivalent Brace of Concrete (Unconfined with Hoops)

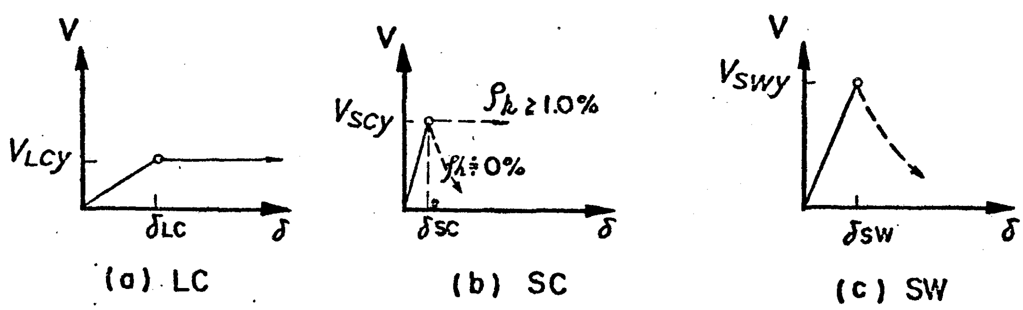


Fig. 11 Story Shear Force - Displacement Relationships of Aseismic Elements

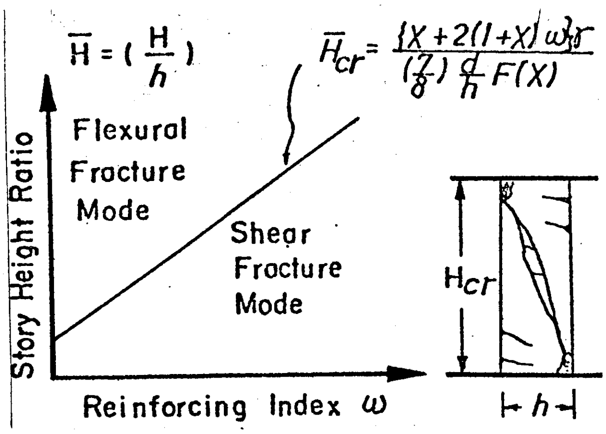


Fig. 12

Zoning of fracture modes of columns (LC and SC)

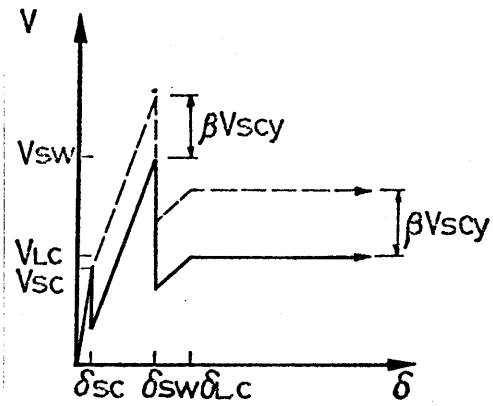


Fig. 13

Story Shear Force - Displacement Relationship of Total Structure

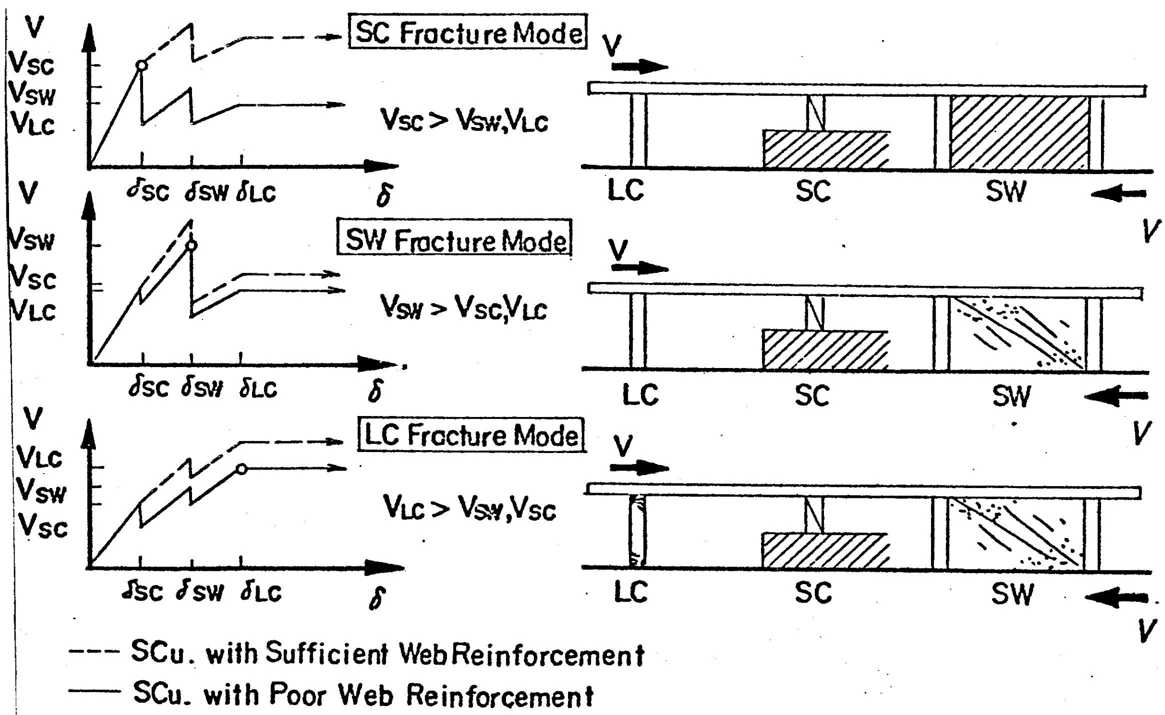
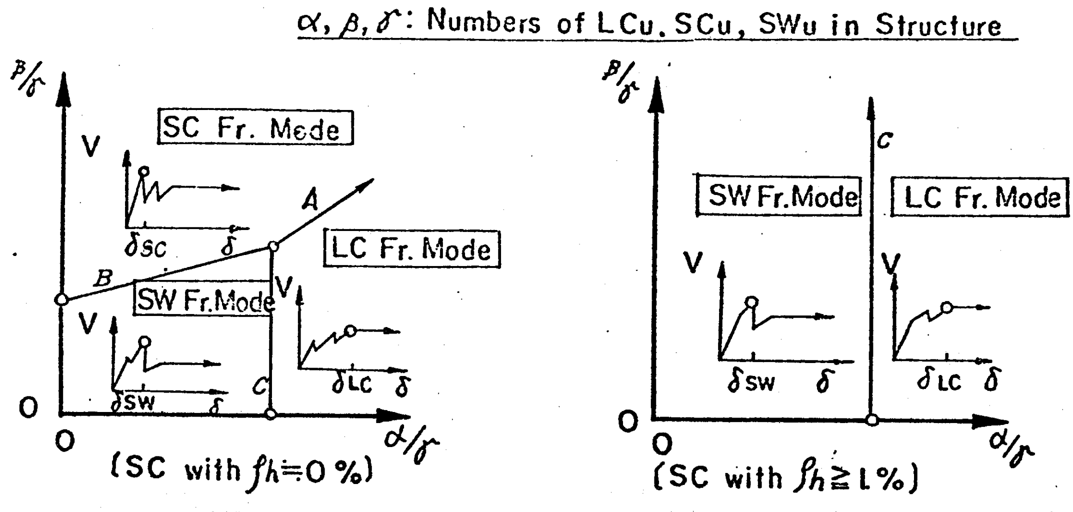
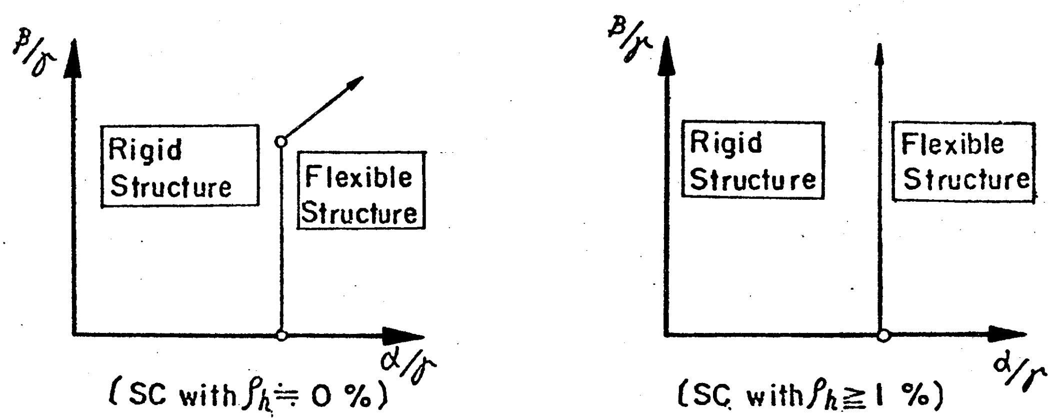


Fig. 14 Classification of Fracture Modes of Structures



(a) (b)

Fig. 15 Zoning of Fracture Modes of Reinforced Concrete Structures



(a) (b)

Fig. 16 Zoning of Rigid and Flexible Types of Reinforced Concrete Structures

CHAPTER 3

RESONANCE CAPACITY CRITERION FOR EVALUATION OF THE ASEISMIC CAPACITY OF REINFORCED CONCRETE STRUCTURES

3-1. Introduction

Recently, in the field of reinforced concrete structures as well as other structures, allowable stress design method has been replaced by ultimate strength or ultimate state design method. From the point of view of aseismic design, however, such a new design method is not sufficient to provide reinforced concrete structures with sufficient resistances against such destructive alternately repeated ground motions as earthquakes. In order to establish a reasonable aseismic design method, it is necessary not only to make clear the mechanical behaviors of reinforced concrete structures subjected to cyclic loadings but to also establish a reasonable method of estimating and applying their cyclic behaviors until fracture.

In order to evaluate the aseismic capacity of reinforced concrete structures, a fundamental new concept is needed for idealizing earthquake motions and consequent structural responses. Therefore, in this chapter, a "RESONANCE CAPACITY" Criterion is proposed. That "Criterion" is derived from the likely steady-state resonance characteristics of the structure. That criterion has an immediate practical significance. It can be used to estimate directly the ultimate capacity of real structures subjected to destructive earthquakes. Further the characteristics of the Criterion can be established directly and simply from the results of experiments on real reinforced concrete structures and members.

3-2. Experimental Data on the Aseismic Characteristics of Reinforced Concrete Aseismic Elements

Experimental Data on Load Bearing Capacity and Hysteretic Damping Capacity

Earthquake forces act on the buildings dynamically as alternately repeated loading. In order to establish reasonable concepts of aseismic characteristics and design guides of reinforced concrete structures, it is necessary to investigate the deformation and fracture behaviors of reinforced concrete members and structures under cyclic loading.

As basic data, in this paper, there are reported the experimental results on the shear resisting capacity and hysteretic damping capacity of reinforced concrete aseismic elements, i.e. shear walls, short columns and long columns, subjected to alternately repeated loadings with constant and incremental lateral deflection amplitudes. Generally, the load-displacement angle hysteresis loops of reinforced concrete members at the first and the later cycles are able to be illustrated as shown in Fig. 1 (a). The story shear force amplitude V_a is defined as that at the specified relative story displacement angle amplitude R_a, and equivalent viscous damping coefficient h_{eq} is given by the following equation (See Fig. 1 (b)):

$$h_{eq} = \frac{1}{4\pi} \frac{\text{Area of Hysteresis Loop}}{\frac{1}{2} V_a R_a} . \qquad (1)$$

Fig. 2 shows the experimental results on the variations of V_a, h_{eq} and $V_a h_{eq}$ to the number of cycles n_c in the case of reinforced concrete shear walls, i.e. SW element with three constant relative story displacement angle amplitudes of $R_a = \pm 0.001, \pm 0.004, \pm 0.015$ (1). Figs.2 (a), (b) and (c) show the $V_a - n_c$, $h_{eq} - n_c$ and $V_a h_{eq} - n_c$ relationships, respectively. (The meanings and usefulness of the value, $V_a h_{eq}$ will be explained later.)

Figs. 3 (a), (b), (c) show the V_a-n_c, h_{eq}-n_c, $V_a h_{eq}$-n_c relationships of reinforced concrete short columns, i.e. SC element with constant displacement angle amplitudes of $R_a = \pm 0.0047, \pm 0.0050, \pm 0.0103$. These cyclic tests of short columns were carried out by Yamada and Yagi (2)(3)(4).

Figs. 4(a), (b), (c) show the V_a-n_c, h_{eq}-n_c, $V_a h_{eq}$-n_c relationships of reinforced concrete long columns, i.e. LC element with constant displacement angle amplitudes of $R_a = \pm 0.0171, \pm 0.0229, \pm 0.0343, \pm 0.0443$ (5)(6)(7).

As for short columns, incremental displacement amplitude tests are carried out, and the results, $V_a - R_a$, $h_{eq} - R_a$, and $V_a h_{eq} - R_a$ relationships are shown in Figs. 5 (a), (b), (c)(4).

From the $h_{eq} - n_c$ relationships in Fig. 2 (b), Fig. 3 (b) and Fig. 4 (b), $h_{eq} - R_a$ relationships of SW, SC, LC elements at the second cycle are plotted in Fig. 6, in order to make clear the effects of R_a upon h_{eq}.

Remarkable Aseismic Characteristics

Looking over Figs. 2, 3, 4, 5, 6, the following remarkable and significant aseismic characteristics of aseismic elements are observed:

(1) As for SW and SC elements characterized by shear failure mode, the deterioration of h_{eq} within an earlier few cycles is very remarkable. The decrease of $V_a h_{eq}$, therefore, is more remarkable than that of h_{eq}. This phenomenon is considered to be caused by the fact that diagonal compressive bracing concrete begins at the second cycle to resist so late due to the closing process of diagonal cracks, formed already at the first cycle, that V - R hysteresis loops become hardening type after the first cycle. (See Figs. 2,3)

(2) On the other hand, as for LC element characterized by flexural yielding type, the decrease of V_a, h_{eq}, $V_a h_{eq}$ are not so remarkable as those of the SW and SC elements. Such characteristics are caused by the relatively stable hysteretic damping capacity of longitudinal reinforcing steel of LC element under the comparatively low axial load ($N = 1/6\ N_0$) (See Fig. 4).

(3) Judging from Fig. 5 (c), under a proper condition of incremental deflection amplitudes, even such a SC element as shows shear failure mode may show constant $V_a h_{eq}$ characteristics to the abscissa R_a. The same behavior is considered to occur even in the case of SW element.

(4) Fig. 6 shows that the hysteretic damping capacity of LC element

increases with the increase of R_a and that, as for SW and SC elements, it is relatively constant and smaller than that of the LC element.

3-3. Fundamental Concept of Aseismic Capacity

Equation of Aseismic Capacity

Generally, aseismic waves are random, so that in the earthquake structural engineering the following two extreme approaches are of temporal use, deterministic and stochastic ones (8). Comparing to the analytical exactness, however, both of them have still many limitations. Even in the results of their analytical operations there still remain many uncertainty factors, so that complicated engineering judgements are needed to apply them to aseismic design. It is the largest defect of them that they are now too analytical to estimate the aseismic characteristics of real structures and structural members directly and experimentally by them.

In this study, therefore, to evaluate the aseismic capacity of real structural members in a simple and sure way, a new concept "RESONANCE CAPACITY" is introduced. This physical quantity is derived from an ultimate state of vibration, i.e. steady-state resonance. By taking the ultimate states of both inputs and outputs, i.e. seismic motions and structural responses into account, an ultimate equilibrium state of energy transformation is able to be reached. "RESONANCE CAPACITY" is directly related to the maximum and substantial ability of energy absorption of structures and structural members, and consequently, its equation is derived as follows:

When one mass oscillation is subjected to sinusoidal acceleration waves as shown in Fig. 7, the differential equation of motion is expressed as

$$m\,\ddot{x} + c\,\dot{x} + f(x,t) = -m\,\alpha_0 \cos(\omega_0 t + \psi)\ . \qquad (2)$$

Assuming that:

(1) the one mass system is to be on the condition of steady-state resonance.

(2) the restoring force function f(x,t) of the system is given as a steady hysteresis loop with constant displacement and story shear force amplitudes, X_1 and V_a, as shown in Fig. 8.

(3) the viscous damping coefficient c is neglected comparing to the hysteretic damping capacity,

(4) the resonant response displacement x is approximately given by

$$x = X_1 \cos \omega_0 t\ , \qquad (3)$$

and integrating the both sides of Eq. 2 over one cycle with respect to x, Eq. 2 is reduced to

$$\oint f(x,t)\ dx = m\alpha_0 X_1 \pi |\sin\psi|\ , \qquad (4)$$

where

$$\oint f(x,t)\ dx = A \text{ (area of hysteresis loop)}, \qquad (5)$$

$$|\sin\psi| \leqq 1\ . \qquad (6)$$

Because of steady-state resonance,

$$\omega_0 = \omega_e = \sqrt{\frac{K_e}{m}} \quad , \tag{7}$$

which is considered to be a very rare but the worst case,

$$|\sin\psi| = 1 \quad . \tag{8}$$

Then Eq. 4 becomes

$$A = m\,\alpha_0 X_1 \pi \quad . \tag{9}$$

Using equivalent viscous damping coefficient h_{eq} (See Eq. 1) and weight of mass W, which are expressed as follows:

$$h_{eq} = \frac{1}{4\pi} \frac{A}{\frac{1}{2} V_a X_1} \quad , \tag{10}$$

$$W = m\,g \quad , \tag{11}$$

finally, Eq. 4 is reduced to

$$\frac{\alpha_0}{g} W = 2\, V_a\, h_{eq} \quad . \tag{12}$$

Eq. 12 has the same physical meaning as the equation,

$$\frac{\text{(forced vibration level)}}{2 \times \text{(viscous damping coefficient)}} = \text{(response level)}$$

in the case of the steady-state resonance of visco-elastic one-mass-system subjected to sinusoidal forced vibration.

The left side of Eq. 12 indicates the acceleration level of forced vibrations, and the right side of Eq. 12 is able to be regarded as the critical response value of members and structures. From this point of view, here, Eq. 12 is proposed as a criterion equation of aseismic capacity and safety of reinforced concrete members and structures. Namely, the value of $2V_a h_{eq}$ nominated here RESONANCE CAPACITY is used as a Criterion of the aseismic capacity, and comparison of it with the left hand side of Eq. 12 makes it possible to estimate the aseismic safety.

Earthquake Resisting Factors

Assuming that earthquake loading consists of such sinusoidal cyclic forces with the maximum constant acceleration amplitude α_0 and period T_0 as make an oscillator to resonate, the oscillator may response with constant RESONANCE CAPACITY and the total number of cycles of oscillation is nearly equal to that of the given cyclic forces. In such a case, the deflection amplitude of LC element is considered to be approximately constant, and that of SC and SW elements is incremental (See Figs. 4(c), 5(c)). Therefore, when deflection angle amplitude R_a or the number of cycles n_c of aseismic elements reaches a critical value, R_{aB} or n_B, the aseismic elements will collapse.

Consequently, when the earthquake-resisting factors of aseismic elements, i.e. $V_a h_{eq}$, R_{aB} and n_B, are given experimentally (such as shown in Figs. 2, 3, 4, 5, 6) or analytically, the maximum aseismic capacity of them is able to be evaluated clearly.

Response V_a - h_{eq} - T_e Interaction

As for LC element, if the left side $\frac{\alpha_0}{g}W$ of Eq. 12 is constant, the relation between V_a and h_{eq} is given as follows:

$$V_a \propto \frac{1}{h_{eq}}, \tag{13}$$

and h_{eq} increases with the increase of R_a (See Figs. 4(b), 6).

As for SW and SC elements, even if $V_a h_{eq}$ is considered to be constant, V_a also becomes nearly constant:

$$V_a \fallingdotseq \text{const.}, \tag{14}$$

because the probable h_{eq} of SW and SC elements is so smaller than that of LC elements (See Fig. 6) that it is able to be considered to be constant from technological point of view.

Considering only resonant state, V_a is independent of T_e which is the natural period of equivalent linear system and given by the following expression:

$$T_e = 2\pi\sqrt{\frac{W R_a H}{g V_a}}. \tag{15}$$

Generally, real earthquakes with destructive acceleration amplitudes are considered to have such a probability distribution to period as shown in Fig. 9, so that, as response value, V_a also has a distribution analogous to it.

From these considerations described above, response V_a - h_{eq} - T_e interaction is able to be illustrated simply as shown in Fig. 10, in which SW and SC elements belong to the range of h_{eq} from 0 to h_{cr} and LC element belongs to the range of h_{eq} more than h_{cr}, and the maximum possibility of the steady-state resonant vibration is considered to occur in the range of T_e from 0 to T_{cr} and the possibility of steady-state forced vibration in the range of T_e more than T_{cr}, where h_{cr} and T_{cr} are critical values which should be decided on those bases of engineering judgements.

Judging from the fracture modes of reinforced concrete structures which are already explained in the previous chapter, the reinforced concrete structures with SW and SC or LC fracture modes are able to be considered to have the same aseismic characteristics of SW, SC or LC element itself, so that the idea of Fig. 10 is valid not only for the aseismic elements but also for reinforced concrete structures. Consequently, rigid structures belong to the range $0 < h_{eq} < h_{cr}$ (See Fig. 6), and flexible structures to the range $h_{cr} < h_{eq}$. That is, the former resists against earthquakes through shear resisting capacity and the latter through hysteretic damping capacity.

As complemental data, the ratios β of the maximum response acceleration to the maximum ground acceleration of one-mass linear system, subjected to several real earthquakes, are shown in Figs. 11 and 12 (9)(10), the abscissas of which are viscous damping coefficient h and natural period T, respectively. As for Fig. 11, the ordinate is the maximum ratio β_{max} within all range of T. It is very interesting that these figures show the same tendency as Fig. 10. Judging from these figures, 0.15 and 1.5 sec may be used as the critical values h_{cr} and T_{cr}, respectively, and the maximum value of β may be about 10/3. When earthquake resisting factors, V_a, h_{eq}, T_e of reinforced concrete aseismic elements and/or structures are given, the estimation of the aseismic safety and the aseismic design of them may be easily performed by relating the factors to the critical values of Eq. 12. The application examples of these ideas to real structures will be discussed in the later chapters.

3-4. Concluding Remarks

In order to establish a fundamental new evaluation method for the aseismic capacity of reinforced concrete structures with universal validity in physical meanings, "RESONANCE CAPACITY" is proposed as a basis of estimation criterion.

Based upon the experimental results of the Resonance Capacities and fatigue behaviors of reinforced concrete structural members subjected to cyclic loadings, the aseismic characteristics of entire reinforced concrete structures are discussed on. As a result, the followings are clarified:

(1) As for hysteretic damping capacity, SC and SW elements under cyclic loadings show very remarkable deterioration after the first cycle. On the other hand, LC element shows comparatively steady behavior even up to the collapse (See Figs. 2, 3, 4).

(2) When one-mass oscillator is subjected to sinusoidal forced vibration and reaches steady-state resonance, which may be the most dangerous case of possible earthquakes, the criterion equation of aseismic capacity is derived from the equilibrium of energy as indicated by Eq. 12.

(3) Under the condition of constant RESONANCE CAPACITY $2V_a h_{eq}$, LC

element responses with relatively constant V_a, R_a, and SC and SW elements with relatively constant V_a, h_{eq}. Furthermore, considering the period distribution of earthquakes, their response V_a - h_{eq} - T_e interaction is able to be illustrated as shown in Fig. 10. Consequently, SC and SW elements and rigid structures, i.e. $(0 < h_{eq} < h_{cr})$ resist against earthquakes through V_a, and the resistance of LC element or flexible structures, i.e. $(h_{cr} < h_{eq})$ may increase with the increase of h_{eq}.

(4) The response acceleration spectra to natural period and viscous damping coefficient are shown in Figs. 11, 12. The outlines of these spectra are so analogous to those of the V_a - h_{eq} - T_e interaction that the fundamental concepts on the aseismic capacity and safety of reinforced concrete structures may be clarified and made useful by the comparison between them.

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NOTATION

A = area of hysteresis loop of restoring force function $f(x, t)$

c = viscous damping coefficient

$f(x,t)$ = restoring force function

h_{cr} = critical viscous damping coefficient

h_{eq} = equivalent viscous damping coefficient

K_e = equivalent linear stiffeness

m = mass

N = axial force

N_o = ultimate strength of centrally loaded columns $(= (1 + 2\omega)f'_c bh)$

n_c = number of cycles

n_B = number of cycles until fracture

R = relative story displacement angle

R_a = amplitude of R

T_o = period of earthquakes

T_e = equivalent natural period of members

t = time

T_{cr} = critical period

V = story shear force

V_a = amplitude of V

W = aseismic weight of structures, weight of mass

X = axial load ratio, $(=N/N_o)$

X_1 = displacement amplitude of hysteresis loops

x = lateral displacement of one-mass oscillator

$\ddot{z}_G$ = acceleration of sinusoidal ground motions

SW = shear wall elements

SC = short column elements

LC = long column elements

α_o = acceleration amplitude of sinusoidal ground motions

β = amplification factor of response acceleration

δ = relative story displacement

ψ = difference of phase angle

ω_o = circular frequency of earthquake waves

ω_e = equivalent linear natural circular frequency

Supplements

RCW : specimen mark of shear wall (1)

RC:C1B : specimen mark of long column (6,6,7)

RC:C1Q : specimen mark of short column (2,3,4)

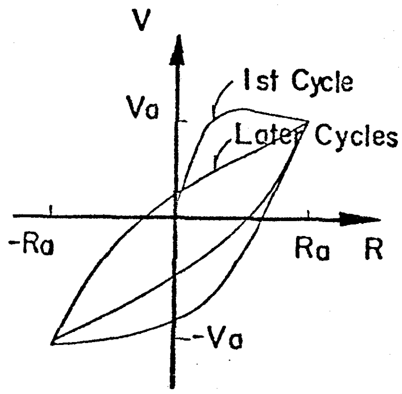

Fig. 1(a) Hysteresis Loops of Story Shear Force and Displacement Angle

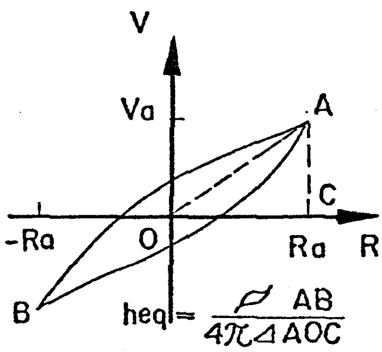

Fig. 1(b) Steady-State Hysteresis Loop

(Equivalent viscous damping coefficient, h_{eq})

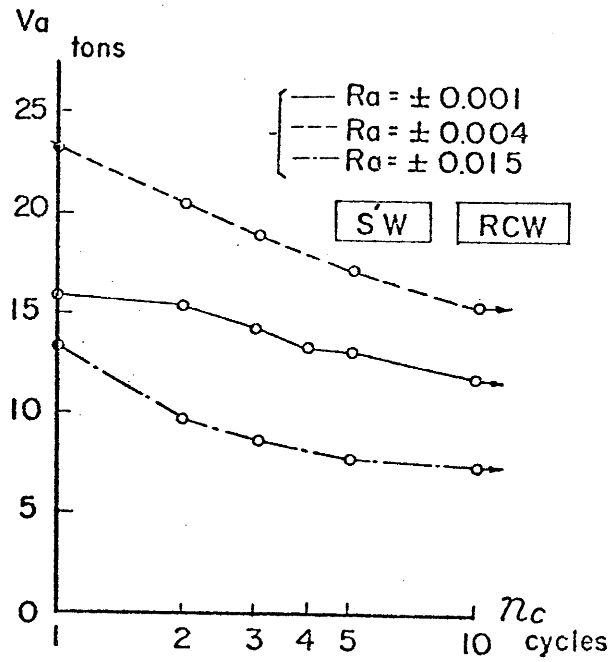

Fig. 2(a) Story Shear Force Amplitude-Number of Cycles Relationships of SW elements (Tests)[1]

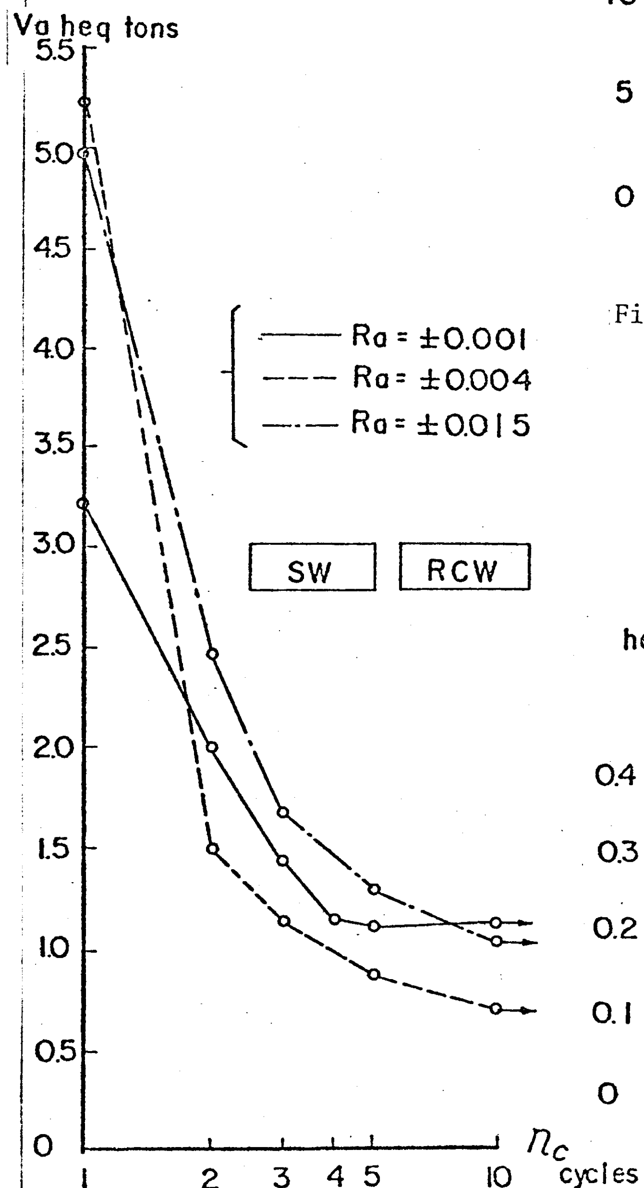

Fig. 2(c) Resonance Capacity-Number of Cycles Relationships of SW elements (Tests)

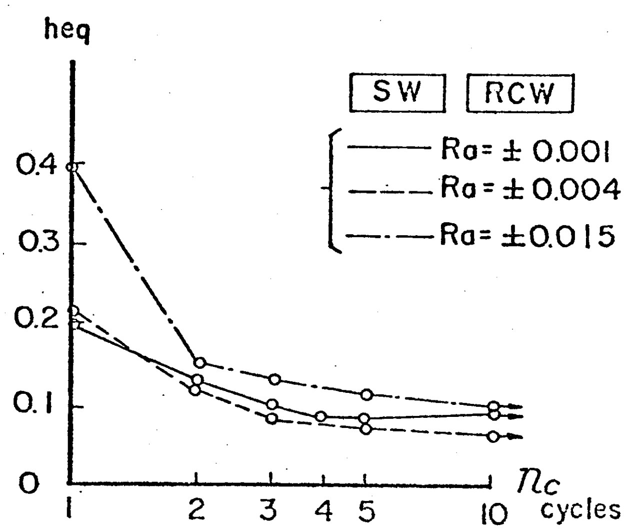

Fig. 2(b) Equivalent Viscous Damping Coefficient-Number of Cycles Relationships of SW elements (Tests)

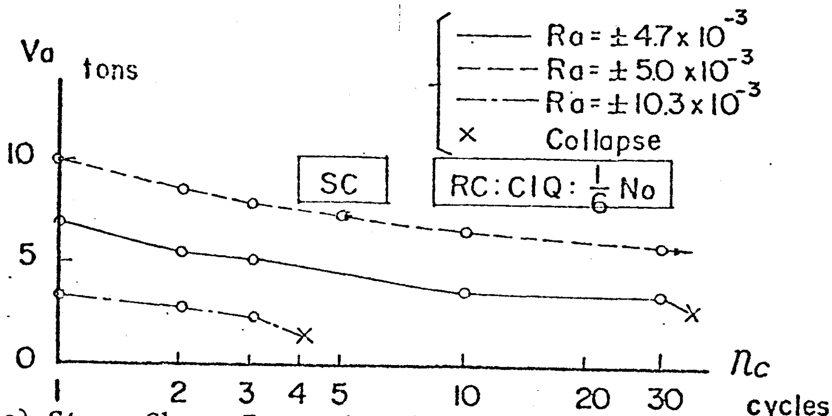

Fig. 3(a) Story Shear Force Amplitude - Number of Cycles Relationships of SC elements (Tests)[2,3,4]

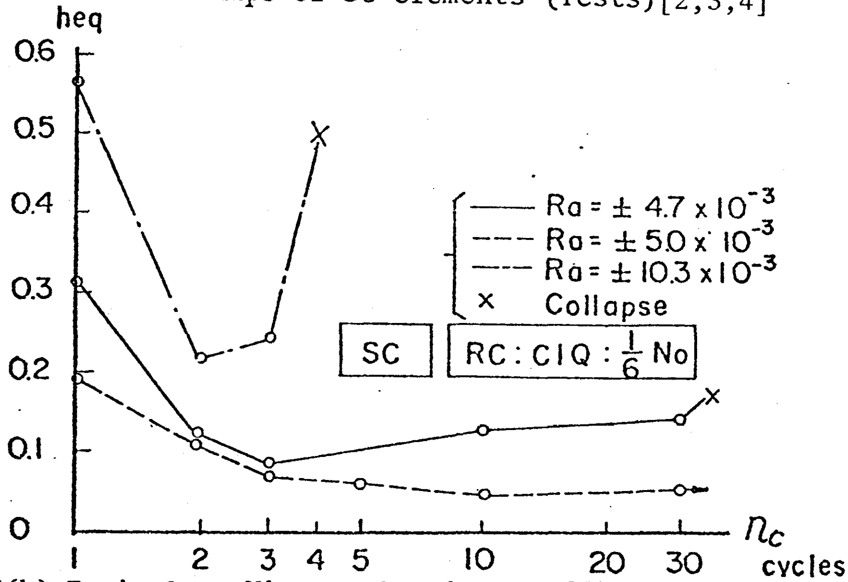

Fig. 3(b) Equivalent Viscous Damping Coefficient - Number of Cycles Relationships of SC elements (Tests)

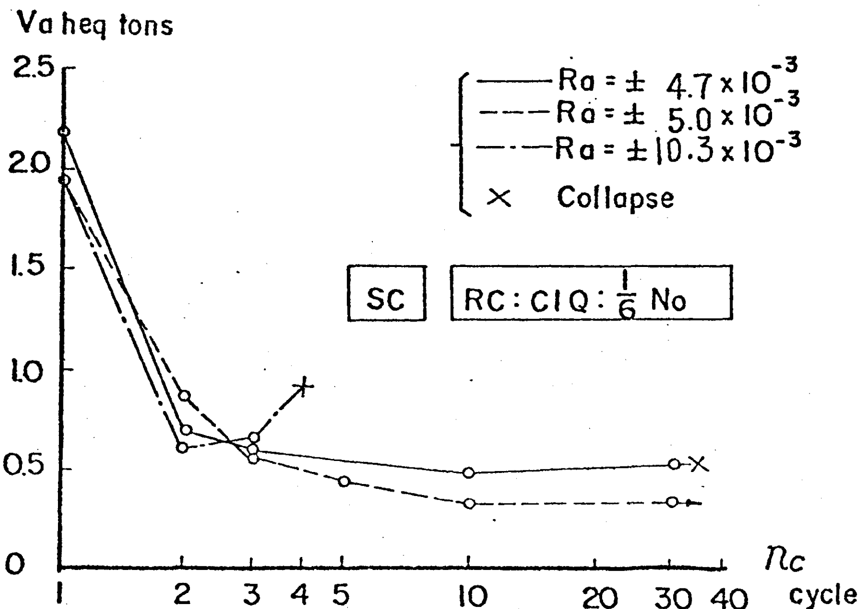

Fig. 3(c) Resonance Capacity - Number of Cycles Relationships of SC elements (Tests)

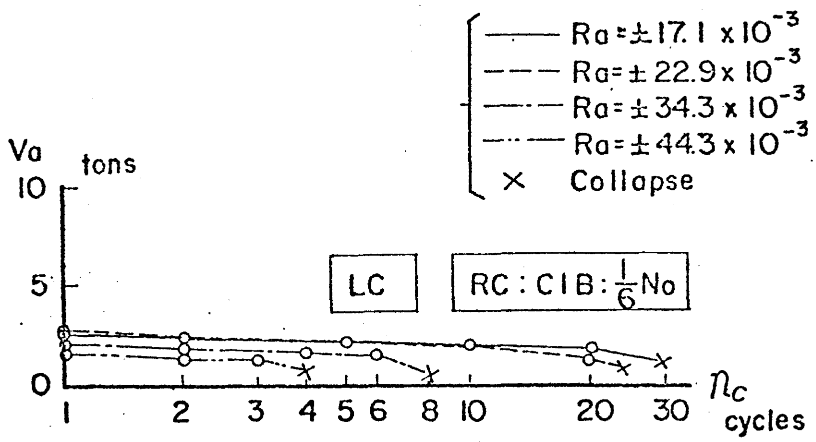

Fig. 4(a) Story Shear Force Amplitude - Number of Cycles Relationships of LC elements (Tests) [5,6,7]

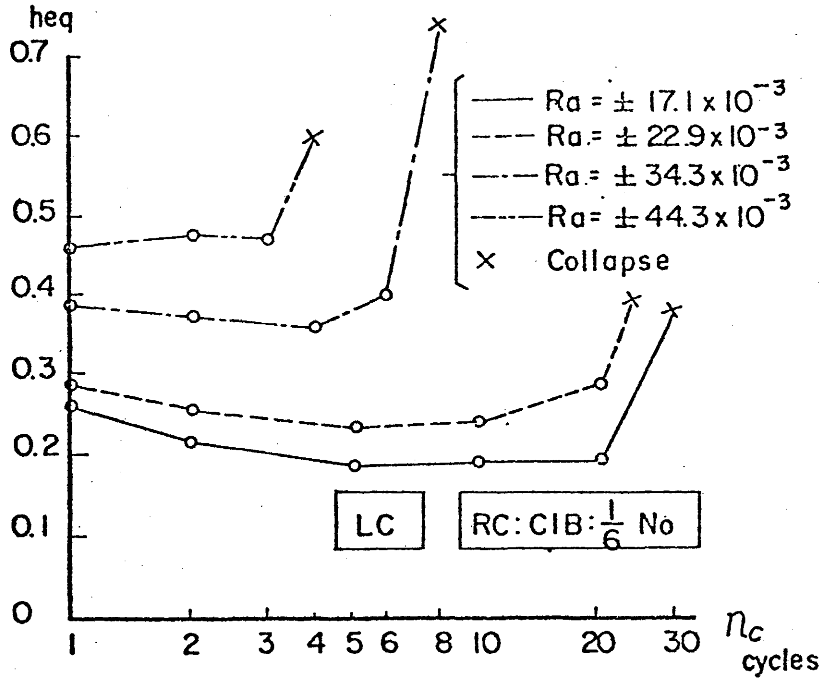

Fig. 4(b) Equivalent Viscous Damping Coefficient - Number of Cycles Relationships of LC elements (TESTS)

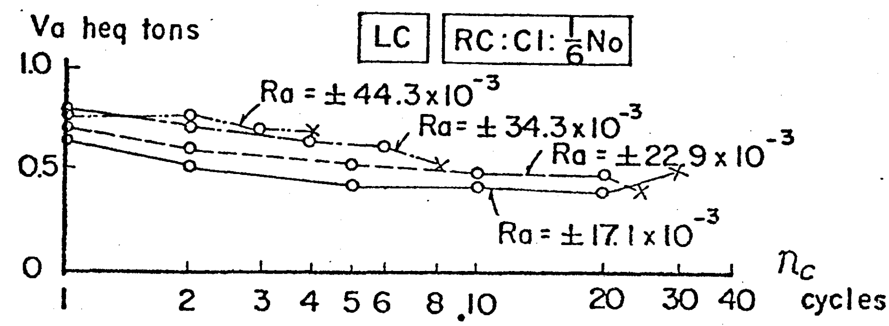

Fig. 4(c) Resonance Capacity - Number of Cycles Relationships of LC elements (Tests)

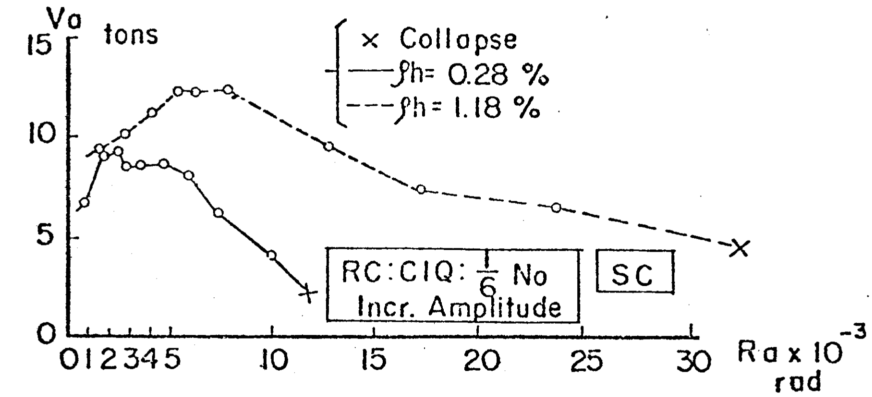

Fig. 5(a) Story Shear Force Amplitude - Incremental Displacement Angle Amplitude Relationships of SC Elements (Tests)[2,3,4]

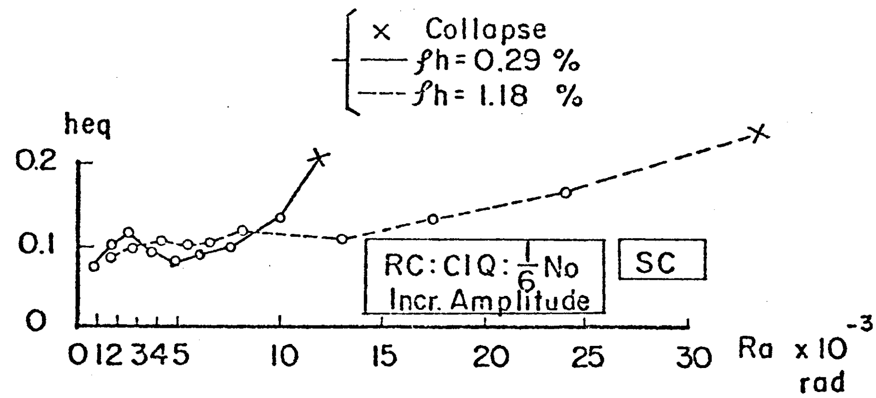

Fig. 5(b) Equivalent Viscous Damping Coefficient - Incremental Displacement Angle Amplitude Relationships of SC Elements (Tests)

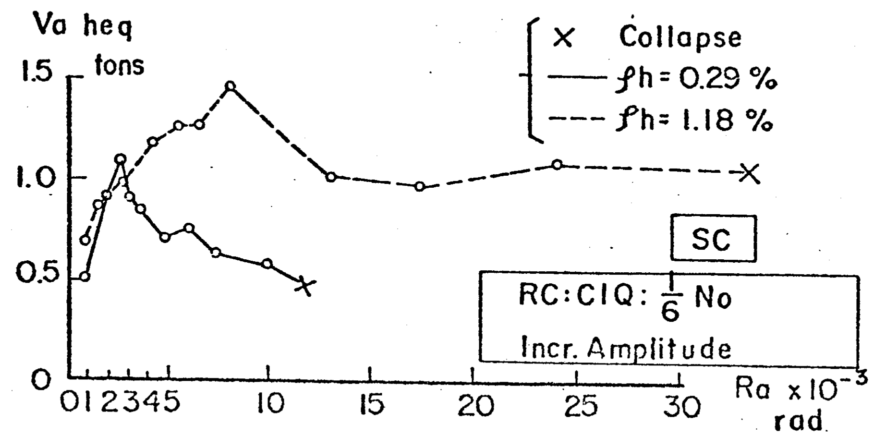

Fig. 5(c) Resonance Capacity - Incremental Displacement Amplitude Angle Amplitude Relationships of SC Elements (Tests)

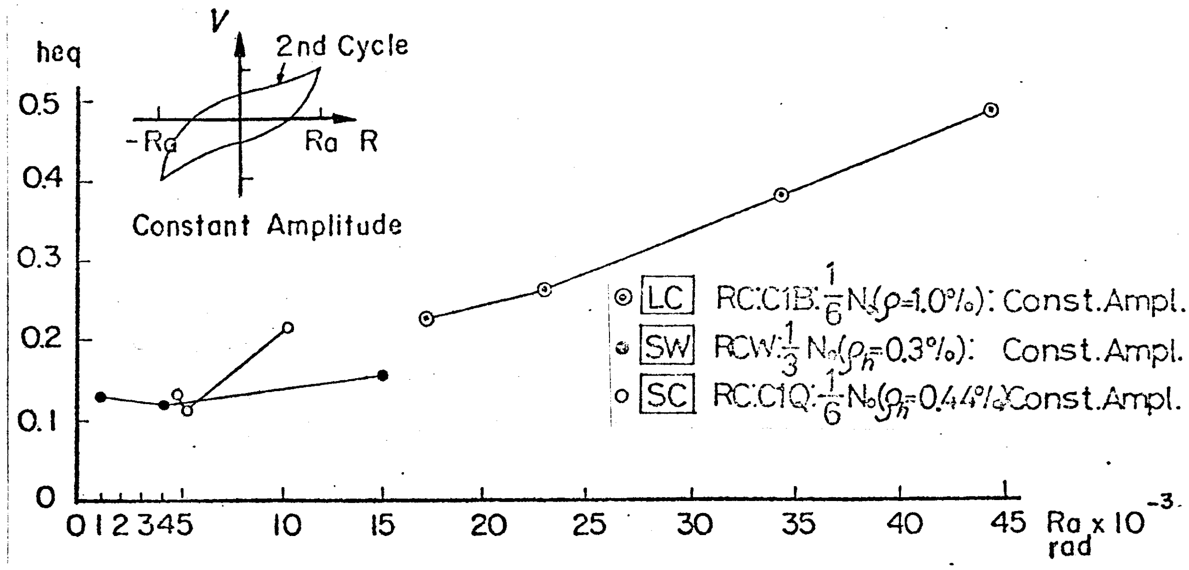

Fig. 6 Equivalent Viscous Damping Coefficient - Constant Displacement Angle Amplitude Relationships of LC, SC, SW Elements (Tests)

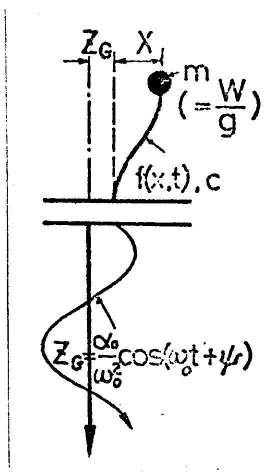

Fig. 7 One-Mass Oscilator

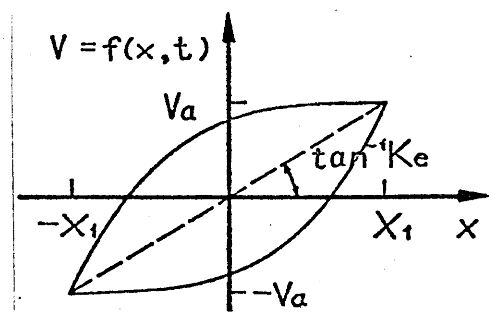

Fig. 8 Restoring Force Function f(x,t) at Steady-state Resonance

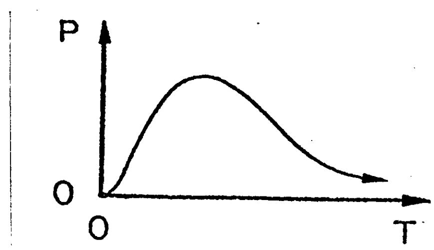

Fig. 9 Probability Density Function of Earthquakes with Destructive Acceleration Amplitudes

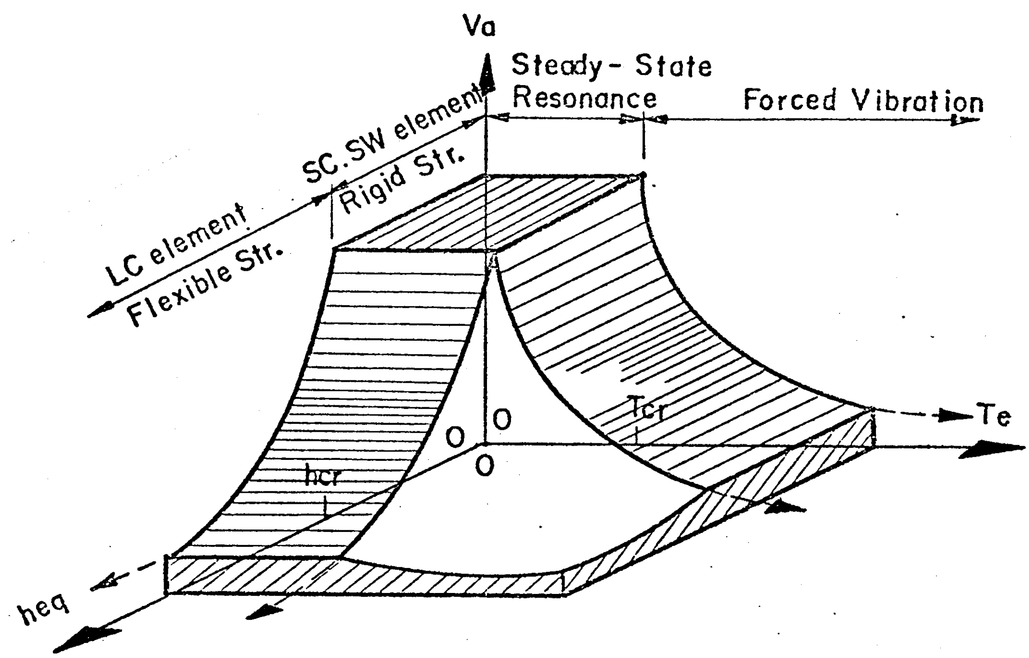

Fig. 10 Response Interaction Surface between Story Shear Force Amplitude, Equivalent Viscous Damping Coefficient, Equivalent Natural Period

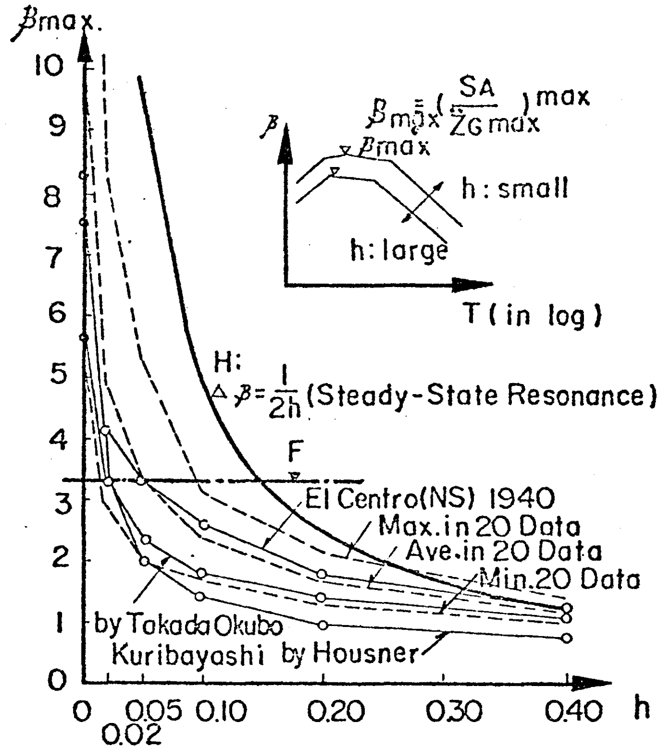

Fig. 11 Shear Force Spectra to Viscous Damping Coefficient

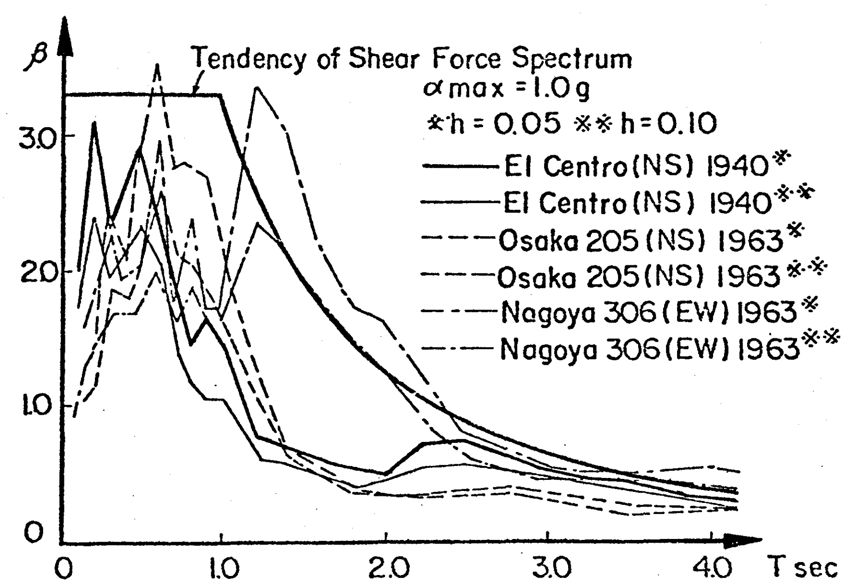

Fig. 12 Shear Force Spectra to Natural Periods
(by SERAC [10])

CHAPTER 4

REINFORCED CONCRETE STRUCTURES WITH AND WITHOUT SHEAR WALLS

4-1. Introduction

In Chapter 2, reinforced concrete structures were considered to be composed of three kinds of aseismic elements, i.e., short columns, long columns and shear walls, and then they were divided into two aseismic types, i.e., rigid and flexible structures. Finally, on the basis of the hysteretic damping capacity of such aseismic elements under cyclic loading, fundamental concepts of the aseismic capacity of reinforced concrete rigid and flexible structures were established. From the technological point of view, only fundamental and abstract concepts are of little use, so that it is necessary to indicate concrete and quantitative applications to real reinforced concrete structures.

The purposes of this chapter are to show concrete procedures of the application of such principles to real medium or low rise reinforced concrete structures and to clarify the aseismic capacity and safety of them quantatively. The method of case study may be the most effective to attain these objects, so that calculations are carried out on the two types of reinforced concrete structures, i.e., rigid and flexible types.

Of course, the fundamental ideas on the aseismic capacity reported in Chapter 3 are not necessarily detailed aseismic design guides. In application of them to the estimation of the aseismic capacity and safety of real reinforced concrete structures, therefore, many assumptions and idealizations must be used. The main significance of this study is able to be considered to exist in the processes of the applications and in the outlines of the aseismic capacity and safety of reinforced concrete structures which are evaluated finally.

4-2. Aseismic Capacity of Rigid Structures

Case Study on The Classification of Standard-Type Reinforced Concrete Structures

Aseismic units-- A reinforced concrete structure is considered to be built as a combination of three aseismic units, LCu, SCu and SWu, which correspond to the aseismic elements, long columns, short columns and shear walls described in Chapter 2, respectively. These aseismic units and their deformation modes are shown in Fig. 1, where each aseismic unit is assumed to carry a unit slab at its top end. The applied values for this examples which are able to be considered to be the most popular and general ones in Japan, are as follows (see Figs. 1 and 2):

L=600cm(236in), H=300cm(118in), b x h=60cm x 60cm(23.6in x 23.6in) As=As'=23.3cm^2(3.6in^2), t=20cm(7.9in), d'/h=d_s/h=0.1, and then $\rho'=\rho$=0.65%, H/h=5.

As for materials, f_c' =200kg/cm^2(2840psi), f_y=4000kg/cm^2(57.0ksi), ${}_c\varepsilon_m$=2.0x10^{-3}, χ =4/3, τ_y/f_c' =0.25, and then $\omega=\omega'$=0.13, G_c/f_c'=450.

Load-displacement relationships-- The story shear force (V) - relative story displacement (δ) relationships of the aseismic units are able to be calculated by using the assumptions and equations in Chapter 2, and they are shown in Fig. 3. By summing the shear forces of the aseismic units at the same displacement, V-δ relationships of reinforced concrete structures which are composed of α-pieces of LCu, β-pieces of SCu and γ-pieces of SWu are able to be given by the following equations:

$$V_{LC} = \alpha V_{LCy} + (\beta V_{SCy}), \tag{1-a}$$

$$V_{SC} = \alpha V_{LCy} (\delta_{SC}/\delta_{LC}) + \beta V_{SCy} + \gamma V_{SWy}(\delta_{SC}/\delta_{SW}), \tag{1-b}$$

$$V_{SW} = \alpha V_{LCy} (\delta_{SW}/\delta_{LC}) + (\beta V_{SCy}) + \gamma V_{SWy}, \tag{1-c}$$

where the value within parentheses is able to be taken into account, when SCu has sufficient shear reinforcement.

Classification of fracture modes-- By means of the classification method of fracture modes of reinforced concrete structures which are composed of α-pieces of LCu, β-pieces of SCu and γ-pieces of SWu, the fracture modes are able to be zoned in the $\alpha/\gamma - \beta/\gamma$ plane as shown in Fig. 4. When SCu has poor shear reinforcement ($\rho_h \fallingdotseq 0\%$), there exist three regions of LC, SC and SW fracture modes (see Fig. 4a), and when SCu has sufficient shear reinforcement ($\rho_h > 1\%$), there exist two regions of LC and SW fracture modes (see Fig. 4b). The boundary lines

A, B and C are able to be drawn by the following expressions, $V_{LC}=V_{SC}$, $V_{SC}=V_{SW}$ and $V_{SW}=V_{LC}$, respectively.

<u>Wall ratios</u>-- When wall ratio is defined as the ratio of the cross sectional area of shear walls to the area of floor, the wall ratio w of the reinforced concrete structures composed of three kinds of aseismic units is expressed by the following equation:

$$w = \frac{tL\gamma}{L^2(\alpha+\beta+2\gamma)} = \frac{20\text{x}600}{36(\alpha/\gamma+\beta/\gamma+2)}\text{x}10^{-4}\ . \qquad (2)$$

The wall ratios w, therefore, are drawn as contourlines in $\alpha/\gamma - \beta/\gamma$ plane such as shown in Fig. 5.

<u>Natural period</u>--In order to calculate natural period T, the followings are assumed:

(1) Reinforced concrete structures consist of one-story aseismic elements and compose one-mass oscillators.

(2) The distributed weight of the mass W_o is assumed to be 1 t/m^2 (1.42psi).

(3) The rigidity K of the oscillator is given as the gradient of the line between the origin and the maximum shear force point whose displacement decides fracture mode.

Consequently, the natural period T is nearly expressed by the following equation:

$$T = \frac{1}{5}\sqrt{\frac{W}{K}}\ , \qquad (3)$$

where W is the weight of mass and is given by

$$W = W_0 L^2(\alpha+\beta+2\gamma)\ , \qquad (4)$$

and K is given by the following equations:

$$K = V_{LC}/\delta_{LC}, \text{ for LC fracture mode,}$$
$$K = V_{SC}/\delta_{SC}, \text{ for SC fracture mode,} \qquad (5)$$
$$K = V_{SW}/\delta_{SW}, \text{ for SW fracture mode.}$$

The natural periods T, therefore, are able to be illustrated as contourlines in $\alpha/\gamma - \beta/\gamma$ plane such as shown in Fig. 6. These contourlines become, naturally, discontinuous on the boundary lines between fracture modes due to jumping of the characteristic values of displacement, $\delta_{LC}, \delta_{SC}, \delta_{SW}$. In each region of fracture modes, T is limited within a range such as indicated in Fig. 6. As for structures with several stories, their natural periods are able to be approximately given by multiplying those of one-story structures by the number of stories.

Critical number of stories-- As the most simplified example, when reinforced concrete structures are composed of several stories of similar aseismic units and subjected to lateral force, which is equal to the dead load W, there must exist the critical number of stories n_{cr}, which is given as follows:

$$n_{cr} = \frac{V_{LC}}{36(\alpha+\beta+2\gamma)}, \text{ for LC fracture mode,}$$
$$n_{cr} = \frac{V_{SC}}{36(\alpha+\beta+2\gamma)}, \text{ for SC fracture mode,} \qquad (6)$$
$$n_{cr} = \frac{V_{SW}}{36(\alpha+\beta+2\gamma)}, \text{ for SW fracture mode.}$$

the critical numbers of stories n_{cr} are also able to be illustrated by means of such contourlines as shown in Fig.7. Although n_{cr} is continuous on the boundary lines of fracture modes, n_{cr} is also limited within a range such as shown in Fig. 7.

Aseismic Characteristics of Reinforced Concrete Structures

In the case study mentioned above, fracture modes, wall ratios, natural periods and the critical number of stories of reinforced concrete structures are able to be illustrated simultaneously in $\alpha/\gamma - \beta/\gamma$ plane such as shown in Fig. 8.

According to the response V_a-h_{eq}-T_e interaction of reinforced concrete structures and to the general tendencies of acceleration spectra shown in Chapter 3, the aseismic characteristics shown in Fig. 8 are effective and useful to estimate the aseismic capacity and to judge the aseismic safety of medium or low rise reinforced concrete structures with SC and SW fracture modes, i.e., rigid structures, because of the following things:

(1) Before shear failure the natural period of rigid structures even with about five stories is less than 1.0 sec. (see Fig. 6), so that the base shear coefficient q_B of such structures may become about 1.0 as the product of the amplification factor of response acceleration β (=3.33, see Fig. 12 in Chapter 3) and the probable maximum ground acceleration of earthquakes α_0 (=0.3g; g is the acceleration of gravity).

(2) Such rigid structures have no deformation ductility after shear fracture and have little damping coefficient (for example, h_{eq} is less than 0.15 in Fig. 6 in Chapter 3), so that they have to resist against earthquakes through story shear capacity (see Fig. 10 in Chapter 3).

Consequently, using Fig. 8, the aseismic design and the estimation of the aseismic capacity and safety of medium or low rise reinforced concrete structures with shear walls become possible. Judging from Fig. 8, wall ratio w is the most significant factor on which the aseismic capacity and safety of such rigid structures depend.

Comparison with Real Cases of Damage

The significance of wall ratio w is also shown in Fig. 9 which was reported by Prof. Shiga, et al (1). This figure presents the four-ranked damage cases of 3-story reinforced concrete structures at the Tokachi-Oki Earthquake (May 16, 1968, Japan). The ordinate and abscissa in Fig. 9 indicate homogeneous story shear stress and wall ratio, respectively. This figure shows that there exist little high or middle grade cases of damages which is marked by X or ● in the range of w more than 50×10^{-4}. The fact that the contour line $n_{cr}=3$ in SW fracture mode in Fig. 8-a lies within the values of w from 40×10^{-4} to 60×10^{-4} coincides with the tendency of the observed damages in Fig. 9.

4-3. Aseismic Capacity of Flexible Structures

Load-Displacement Hysteresis Loops of Reinforced Concrete Columns

In order to estimate the aseismic capacity of flexible reinforced concrete structures which are composed predominantly of long columns with flexural yielding mode and sufficient hysteretic damping capacity, it is the most basic approach to calculate the steady-state hysteresis loops of story shear force and relative story displacement of long columns subjected to constant axial load and alternately repeated lateral load as double curvature system (see Fig. 10).

Assumptions-- In order to calculate the simplified hysteresis loops of story shear force and relative story displacement of long columns, the followings are assumed:

(1) Hysteresis loops are in steady-state.

(2) Columns are subjected to a little constant axial load (for example, about $(1/6)N_o$; N_o is the ultimate strength of centrally loaded columns).

(3) The displacement amplitudes of columns are sufficient large enough to give sufficient hysteretic damping capacity.

(4) The elastic deflection of columns is calculated by Mohr's theorem, and $P\Delta$ effect is neglected.

(5) The plastic deflection of columns is caused only by the plastic hinges at the top and bottom ends of columns, so that the condition of constant displacement amplitudes corresponds nearly to that of constant curvature amplitudes at the plastic hinges (2) (see Fig. 11).

(6) After the flexural yielding, both the compressive and tensile reinforcements yield, and the compression stress block of concrete is rectangular (see Fig. 12).

(7) When the compressive reinforcement yields before yielding of the tensile reinforcement and closing of the crack of compressive concrete, slipping phenomenon occurs in the hysteresis loops of bending moment and curvature (see Fig. 13).

(8) The effects of the shear force are neglected, and the normal stress distribution remains plane after bending.

Moment-curvature hysteresis loops-- According to the assumptions mentioned above, the moment-curvature hysteresis loops of the cross section of reinforced concrete columns are able to be simplified and idealized as shown in Fig. 13. The characteristic bending moments, M_y and M_s and curvatures, $h\Phi_y$ and $h\Phi_s$ are given as follows:

$$\frac{M_y}{f'_c bh^2} = \frac{1}{2}(1-x_{nl})x_{nl} + \omega(1-d'/h-d_s/h) , \tag{7-a}$$

$$h\Phi_y = \frac{2_s\varepsilon_y}{(1-d'/h-d_s/h)} , \tag{7-b}$$

where $\qquad x_{nl} = N/f'_c bh,$ (8)
and

$$\frac{M_s}{f'_c bh} = (1-K')\,\frac{1-d'/h-d_s/h}{2} , \tag{9-a}$$

$$h\Phi_s = h\Phi_y \frac{M_s}{M_y} , \tag{9-b}$$

where $\quad K' = x_{nl}(1/\omega)-1.$ (10)

<u>Load-displacement hysteresis loops</u>-- According to the assumptions mentioned above and to the simplified moment-curvature relationships, the hysteresis loops of story shear force V and relative story displacement δ are able to be calculated as shown in Fig. 14. The characteristic values of V^B_y, V^B_s, δ^B_y and δ^B_s are given as follows:

$$\frac{V^B_y}{f'_c bh} = 2\,\frac{M_y}{f'_c bh^2}\Big/\frac{H}{h} , \qquad \delta^B_y = H\frac{1}{6}h\Phi_y\cdot\frac{H}{h} , \tag{11}$$

$$\frac{V_s^B}{f'_c bh} = 2\frac{M_s}{f'_c bh^2} / \frac{H}{h}, \quad \delta_s^B = \delta_s^B \frac{V_s^B}{V_y^B}. \qquad (12)$$

Equivalent viscous damping coefficient-- Under cyclic loading, in reality, reinforcing steels show Bauschinger effect, and concrete also shows the similar phenomenon, so that V- δ hysteresis loop of Fig. 14 is able to be idealized into such a figure as shown in Fig. 15. The equivalent viscous damping coefficient h_{eq} of the V- δ hysteresis loop in Fig. 15 is expressed by the following equation (see Eq. 1 in Chapter 3):

$$h_{eq} = \frac{5+9s}{8\pi}\left(1 - \frac{1}{\mu a}\right), \qquad (13)$$

where

$$s = V_s^B / V_y^B, \qquad \mu a = \delta_a / \delta_y^B. \qquad (14)$$

Then h_{eq} is the function of ductility factor μa and increases with the increase of μ_a as shown in Fig. 16.

Fatigue Characteristics of Reinforced Concrete Columns

The aseismic criterion equation

$$\frac{\alpha_o m}{2h_{eq}} = V_a, \quad (15) \quad \text{or} \quad m\alpha_o = 2V_a h_{eq}, \qquad (15')$$

which were introduced in Chapter 3, shows that h_{eq} and necessary V_a are in reciprocal proportion. Considering Eqs. 13, 15, it is concluded that the necessary V_a decreases with the increase of μ_a.

In reality, however, the increase of μ_a is limited by the number of cycles until fracture n_B due to the fatigue characteristics of reinforced concrete columns, i.e., μ_a-n_B relationships.

Assumptions--In order to obtain analytically the μ_a-n_B relationships of reinforced concrete columns, the followings are assumed:

(1) Such flexural mechanisms as we already assumed and idealized to calculate M-Φ and V-δ relationships are valid here, too.

(2) The stress and strain distributions of cross section at a curvature amplitude are assumed as shown in Fig. 12, where $\Phi = \Phi_\alpha$.

(3) The compressive capacity of concrete stress block at the curvature amplitude deteriorates with the increase of the number of cycles as shown in Fig. 17. The deterioration factor ϕ is expressed by the following equation:

$$\phi = 1 - \frac{1}{8} \log_{10} n_c, \tag{16}$$

where n_c is the number of cycles.

Eq. 16 is assumed from the experimental data of plain concrete subjected to repeated loads with constant strain amplitudes shown in Fig. 18, which were given by Yamada and Shimada (3).

(4) The deterioration of the compressive capacity of concrete stress block makes x_{n1} and ε'_s increase.

(5) When ε'_s reaches ${}_c\varepsilon_u$, the cross section of reinforced concrete columns under cyclic bending moments and constant axial loads collapse due to buckling of compressive reinforcing steels, because the columns no longer sustain the specified axial loads.

(6) The longitudinal zone of plastic hinges which produce the plastic deflections of columns is equal to the total depth of their cross section h.

(7) Columns under monotonic loading show flexural yielding on the condition that the shear story height ratio $\overline{H}$ (=H/h) is more than the critical story height ratio H_{cr} which was derived in the preceding chapter(see Eq. 13 in Chapter 2), that is

$$\overline{H} > \overline{H}_{cr} = \frac{\{X+2(1+X)\omega\}\gamma}{(7/8)(d/h)F(X)}, \qquad (17)$$

where

$$F(X) = \sqrt{-0.1X^2+0.09X+0.01}. \qquad (18)$$

<u>Relationships between curvature amplitude and the number of cycles until fracture</u>--Based upon the assumptions mentioned above and Figs. 12, 17, the relationship between the critical curvature amplitude $h\,\Phi_{acr}$ and the number of cycles until fracture n_B is expressed by the following equation:

$$\frac{c\varepsilon u}{x_{n1}/\phi - d_s/h} = \frac{c\varepsilon_u}{\frac{1}{1-(1/8)\log_{10}n_B}\cdot\frac{N}{f'_c bh} - d_s/h} = h\Phi_{acr}, \qquad (19)$$

and its tendency is shown in Fig. 19. Eq. 19 is naturally valid only for the value of $N/(f'_c bh)$ which is so appropriately small that plastic hinges occur at the top and bottom of columns and that the denominator of the left two sides of it is positive.

<u>Relationships between deflection amplitude and the number of cycles until fracture</u>--By means of plastic hinge method the deflection amplitude δ_a is given by the following equation:

$$\delta_a = H\cdot h(\Phi_a - \Phi_y) + \delta_y^B. \qquad (20)$$

By substituting the left side of Eq. 19 for $h\Phi_a$ of Eq. 20, the relationship between deflection amplitude δ_a and the number of cycles until fracture n_B is able to be obtain. If one wants the relationship between ductility factor amplitude μ_a and the number of cycles until fracture n_B, the following expression is able to be used instead of Eq. 20:

$$\mu_a = 1 + \frac{H \cdot h(\Phi_a - \Phi_y)}{\delta_y^B} . \qquad (21)$$

The comparison of the calculated results given by Eq. 19 with the experimental results (5) is shown in Fig. 20. There is a little quantitative difference but good qualitative agreement between them. Figs.21 (a)(b) show the test specimen and raw data (4) from which the test results in Fig. 20 were derived.

Estimation of Aseismic Capacity

If a critical number of loading cycles of predominant waves of earthquakes is given, making it equal to the number of cycles until fracture n_B, a critical deflection amplitude δ_{acr} or a critical ductility amplitude μ_{acr} is able to be derived from Eqs. 19, 20 or Eqs. 19, 21. Consequently, a critical equivalent viscous damping coefficient corresponding n_B is able to be calculated by using Eq. 13. Finally, making V_y^B equal to V_a, the estimation of the aseismic capacity of long columns becomes possible by means of the aseismic criterion equation, i.e., Eq. 15. Even if the values of $V_a h_{eq}$ of columns in a story differ from each other, the summation of them, $\Sigma(V_a h_{eq})_i$, is able to be considered as the RESONANCE CAPACITY of this total story under the condition that all the deflection amplitude of them is identical, because $V_a h_{eq}$ is the function only of load-deflection hysteresis loop area and deflection amplitude.

In this study, although column-yielding type is treated, if the hysteretic damping capacity of beams with slabs is clarified, beam-yielding type should be taken into account. Generally, the hysteretic damping capacity of beams is more than that of columns, the treatment in this study may be considered to be conservative.

Applications to Standard-Type Reinforced Concrete Structures

Assumed conditions--As an example, the aseismic capacity of 4-story reinforced concrete structures without shear walls assumed to belong to column-yielding type is calculated. In order to simplify the problem, calculations are performed only for one column on the ground floor. The sizes and other properties of this column and the mechanical properties of materials used are assumed as follows:

$H = 300$cm(118in), $\overline{H}=H/h=5$, $b \times h=60$cm$\times 60$cm(23.6in$\times$23.6in),

$f'_c = 200$kg/cm^2(2840psi), $f_y=4000$kg/cm^2(57000psi), ${}_s\varepsilon_y=2.0\times 10^{-3}$,

${}_c\varepsilon_u = 4\times 10^{-3}$, $d'/h=d_s/h=0.1$,

Reinforcing index $\omega = 0.40$, 0.35, 0.30, 0.25, 0.20, 0.15,

the weight of equivalent mass $W = 144$ton(317kips)(4 stories x 36 ton/story).

These values are so selected as to fit the most popular and standard ones in Japan.

μ_{acr}-n_B relationships-- By using Eqs. 16, 8, 19, 7, 11, 21, the relationship between the critical ductility amplitude μ_{acr} and the number of cycles until fracture n_B of the assumed column is able to be calculated as shown in Table 1, and given by an approximate straight line in log-log coordinates such as shown in Fig. 22. This relationship is valid for the all assumed columns with different reinforcing indices ω, because $h\Phi_y$ is assumed to be independent of ω (see Eq. 7-b).

Critical ductility amplitude μacr-- If the critical number of loading cycles of predominant earthquakes which the structure under consideration undergoes in its life is about 200 cycles, Fig. 22 shows that the critical ductility amplitude μ_{acr} is about "5" for the assumed column. Judging from Fig. 20 which shows the comparison of the experimental results with analytical values, the calculated value "5" may be overestimated one. In this study, however, it is another significant object to indicate the procedure and method for the estimation of aseismic capacity, so that this value is used for the following evaluation procedures.

Judgement of aseismic safety--By using the critical ductility amplitude "5", the right side of the aseismic criterion equation (Eq. 15'), i.e., RESONANCE CAPACITY, $2V_a h_{eq}$ of the assumed column is shown in Table 2 which indicates the procedures of calculation with equation numbers applied. If the maximum acceleration amplitude ratio to that of gravity of earthquakes, α_o/g, is 0.3, the left side of Eq. 15', i.e., input level becomes 43.2 tons (95.2kips)(=0.3x144tons). By means of interpolation, Table 2 shows that the value of reinforcing index ω more than about 0.24 is necessary to ensure the aseismic capacity of the assumed column.

The evaluation procedures mentioned above are valid only for the case in which plastic hinges occur, that is, the assumed column should belong to flexural yielding type. At the bottom of Table 2, therefore, the critical shear story height ratios $\overline{H}_{cr}$ are calculated in the case that ω =0.40~0.15. Judging from the values of $\overline{H}_{cr}$, the columns with the reinforced index ω less than 0.25 show the flexural yielding type.

Finally, it is able to be concluded that the column only with the reinforcing index value ω of about 0.25 has sufficient aseismic capacity. The base shear coefficient q_B of the assumed 4-story reinforced concrete flexible structure must be assumed to be the value from 0.5 to 0.6 for the aseismic design, where q_B is expressed as follows:

$$q_B = V_a/W = (\alpha_o/g)/2h_{eq}. \qquad (22)$$

4-4. Concluding Remarks

Based upon the fundamental concepts of the aseismic capacity of reinforced concrete structures proposed in Chapter 3, the aseismic capacity and safety of the main two types of real medium or low rise reinforced concrete structures, i.e., rigid and flexible structures are investigated by calculations.

The most typical types of real reinforced concrete structures are adopted as the objects of calculations, and as a result, the followings are concluded.

As for rigid structures which are assumed to be composed of a large number of short column and shear wall units and a small number of long column units, therefore which show shear fracture mode without deformation ductility;

(1) The factors of aseismic characteristics, i.e., fracture modes, wall ratios w and natural periods T are able to be illustrated shown in $\alpha/\gamma-\beta/\gamma$ plane, where α, β and γ are the numbers of pieces of long column u., short column u. and shear wall u., respectively (see Figs. 4, 5, 6, 8).

(2) Judging from the natural periods of medium or low rise structures less than 1 sec. and from the general tendency of response spectrum (Chapt.3) it is able to be considered that the lateral load at earthquakes is nearly equal to the dead load of them.

(3) According to the aseismic concept that rigid structures with little hysteretic damping capacity should resist against earthquakes through story shear force capacity, the maximum number of stories is able to be illustrated by contourlines in $\alpha/\gamma-\beta/\gamma$ plane, too (see Figs. 7, 8).

(4) Wall ratio w proved to be the most significant factor against earthquakes (see Figs. 5, 8) with the evidence of real damaged examples (see Fig. 9).

As for flexible structures which are assumed to be composed only of long column units which show flexural yielding type;

(1) According to the aseismic criterion equation, Eq. 15', the aseismic capacity of flexible structures by RESONANCE CAPACITY, $2V_a h_{eq}$.

(2) V_a, i.e., relative story force amplitude is equal to relative story flexural yielding force V_y^B and given as the function of reinforcing index ω (see Eqs. 7, 11).

(3) h_{eq} is given by the function of relative story displacement

amplitude δ_a (or ductility factor amplitude μ_a) as expressed by Eq. 13.

(4) As fatigue characteristics of reinforced concrete long columns subjected to cyclic lateral loading, the relationships between relative story displacement amplitude δ_a (or ductility factor amplitude μ_a) and the number of cycles until fracture n_B are able to be given analytically by Eqs. 19, 20 (or Eqs. 19, 21) (see Fig. 22).

(5) By applying the number of predominant waves of earthquakes to such relationships, a critical ductility amplitude μ_{acr} is able to be decided.

(6) Finally, the limited value of equivalent viscous damping coefficient h_{eq} is given by Eq. 13.

(7) As a result of the case study on 4-story reinforced concrete structure which is assumed to show column-yielding type, the following values are necessary to ensure sufficient aseismic capacity and safety:

ω = 0.24, for the columns on the ground floor,

q_B = 0.5~0.6, for the base story.

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NOTATION

A_s = area of tension reinforcement

A'_s = area of compression reinforcement

b = width of cross section of columns

d_s = distance from centroid of tension reinforcement to the tensile face of columns

d' = distance from extreme compression fiber to centroid of compression reinforcement

F( ) = fracture criterion function of concrete under combined normal and shear stresses

f_y = yield strength of reinforcement

f'_c = compressive strength of concrete

G_c = modulus of shear of concrete

g = acceleration of gravity

H = story height

$\overline{H}$ = H/h

h = total depth of cross section of concrete

h_{eq} = equivalent viscous damping coefficient

K = stiffeness of aseismic units

K' = slipping factor ($=N/f'_c bh\omega-1$)

L = span length

M = bending moment

M_y = yield moment

M_s = slipping moment

m = mass

N = axial force

N_o = ultimate strength of centrally loaded columns

n_B = the number of cycles until fracture

n_c = number of cycles

n_{cr} = critical number of stories

q_B = base shear coefficient

s = slipping coefficient ($=M_s/M_y$, V_s/V_y)

T = natural period

t = thickness of shear walls

V = story shear force

V_{LC} = V of structures when LC elements show flexural yielding

V_{SC} = V of structures when SC elements show shear yielding

V_{SW} = V of structures when SW elements show shear fracture

V_{LCy} = V of LC elements when they show flexural yielding

V_{SCy} = V of SC elements when they show shear yielding

V_{SWy} = V of SW elements when they show shear fracture

V_y^B = V of columns when they show flexural yielding

V_s^B = V of columns when they show flexural slipping

V_a = story shear force amplitude

w = wall ratio

W = seismic weight

W_o = distributed seismic weight

X = N/N_o

x = lateral displacement of one mass system

x_{nl} = ratio of distance from extreme compression fiber to neutral axis to the total depth of cross section of columns

LCu = long column unit

SCu = short column unit

SWu = shear wall unit

α = number of pieces of LCu

α_0 = acceleration amplitude of sinusoidal ground motion

β = number of pieces of SCu, amplification factor of resp. accel.

δ = relative story displacement

δ_a = amplitude of δ

δ_{LC} = δ at which LCu shows flexural yielding

δ_{SC} = δ at which SCu shows shear yielding

δ_{SW} = δ at which SWu shows shear fracture

δ_y^B = flexural component of δ_y at which columns show flexural yielding

δ_y^S = shear component of δ_y at which columns show shear yielding

γ = number of pieces of SWu, ratio of distance center-to-center of reinforcement on opposite faces of columns to h

${}_c\varepsilon_u$ = ultimate compressive strain of concrete

${}_c\varepsilon_y$ = yielding compressive strain of concrete (idealized)

${}_s\varepsilon_y$ = yielding strain of reinforcement

ω = geometrical ratio of tensile reinforcement

ω' = geometrical ratio of compressive reinforcement

Φ_a = curvature amplitude

Φ_{acr} = critical curvature amplitude

Φ_s = slipping curvature

ϕ = deterioration factor of compressive stress block of concrete

χ = ${}_s\varepsilon_y / {}_c\varepsilon_y$

μ_a = ductility factor amplitude,

μ_{acr} = critical μ_a

Table 1. Critical Ductility Factor Amplitudes

n_B cyc	10^0	10^1	10^2	10^3	10^4	10^5	10^6	10^7	10^8	Eq.
ϕ	1.000	0.875	0.750	0.625	0.500	0.375	0.250	0.125	0.000	16
x_{nl}					0.20					8
$h\Phi_{acr}$ $\times 10^{-3}$	40.0	31.10	24.00	18.18	13.33	9.23	5.71	2.67	0.00	19
$h\Phi_y$ $\times 10^{-3}$					5.00					7
δ_y^B cm					1.25					11
μ_{acr}	9.40	7.24	5.56	4.16	3.00	2.02	1.17	—	—	21

Table 2. Estimation of Aseismic Capacity and Safety

ω	0.40	0.35	0.30	0.25	0.20	0.15	Eq.
M_y/f'_cbh^2	0.400	0.360	0.320	0.280	0.240	0.200	7-a
M_s/f'_cbh^2	0.240	0.200	0.160	0.120	0.080	0.040	9-a
s	0.600	0.556	0.500	0.429	0.333	0.200	11,12,14
μ_{acr}			5.0				—
h_{eq}	0.331	0.318	0.302	0.282	0.254	0.217	13
f'_cbh ton			720				
f'_cbh^2 t-cm			43200				
M_y t-cm	17280	15550	13820	12100	10370	8640	7-a
V_y^B ton	115.2	103.7	92.1	80.7	69.1	57.6	11
$2V_y^Bh_{eq}$ ton	76.3	66.0	55.6	45.5	35.1	25.0	—
α_oW/g ton			43.2				—
Judgement	O	O	O	O	X	X	15'
N_o/f'_cbh	1.8	1.7	1.6	1.5	1.4	1.3	—
$X(=N/N_o)$	0.1111	0.1176	0.1250	0.1333	0.1429	0.1538	18
F(X)	0.1370	0.1386	0.1403	0.1422	0.1443	0.1465	18
$\overline{H}_{cr}$	7.42	6.60	5.79	5.00	4.22	3.47	17
real $\overline{H}$			5.0				—
Frac. Modes	S	S	S	S,F	F	F	17

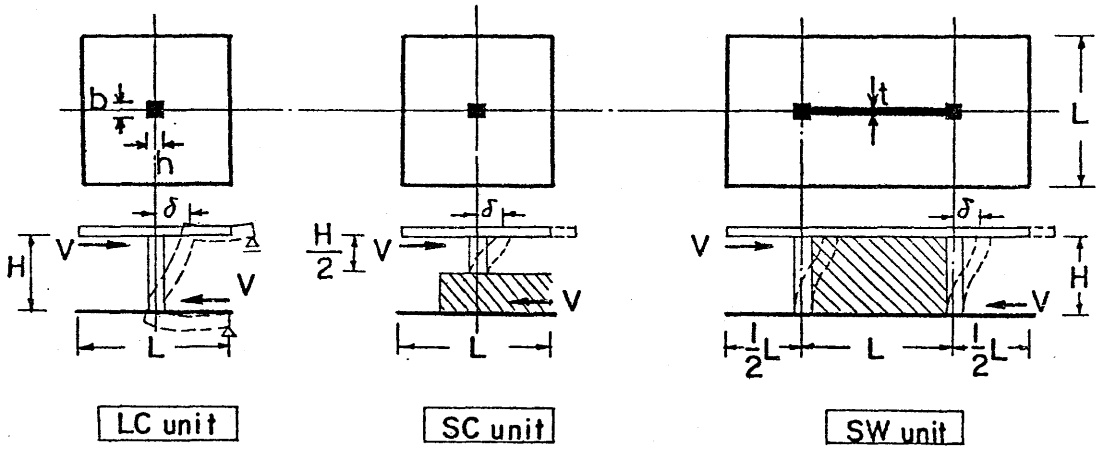


Fig. 1 Aseismic Units and their Deformation Modes

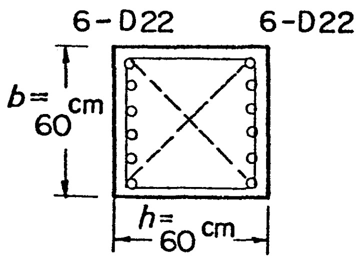
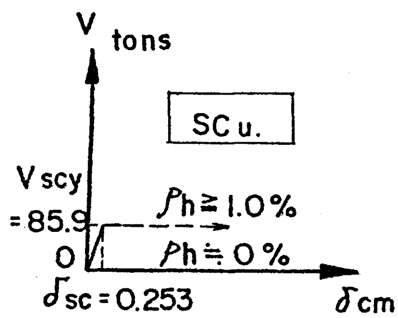
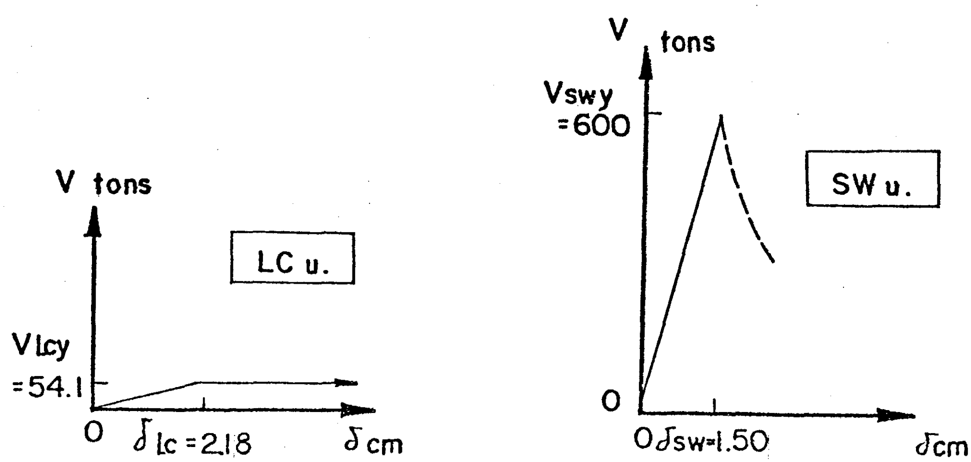


Fig. 2 Cross Section of Assumed Reinforced Concrete Column



(b)



(a) (c)

Fig.3 Story Shear Force-Displacement Relationships of Aseismic Units

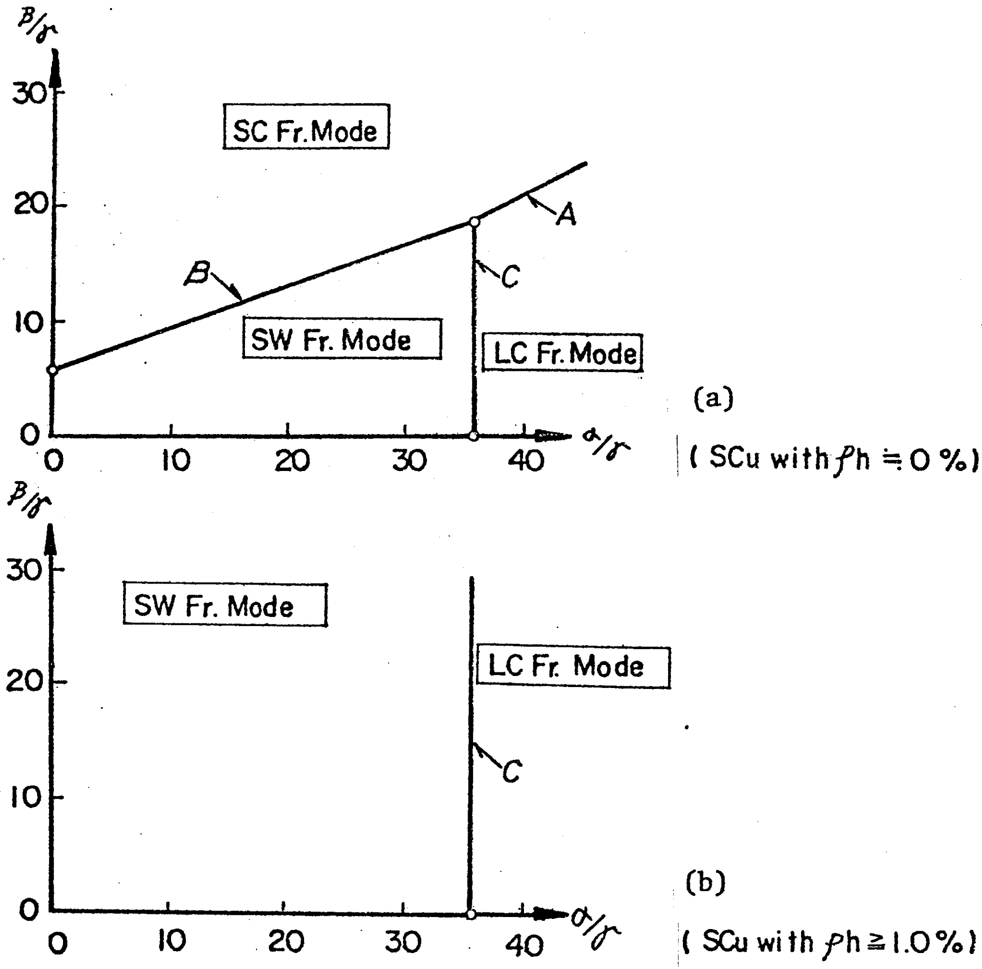


Fig.4 Fracture Modes of Reinforced Concrete Structures

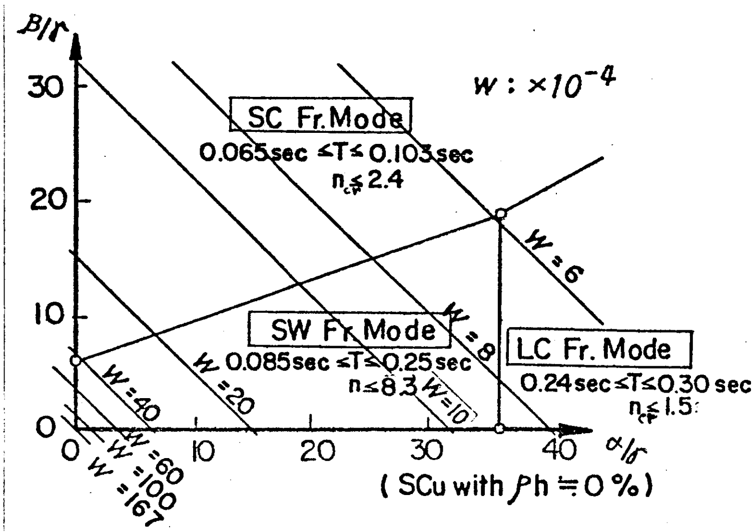
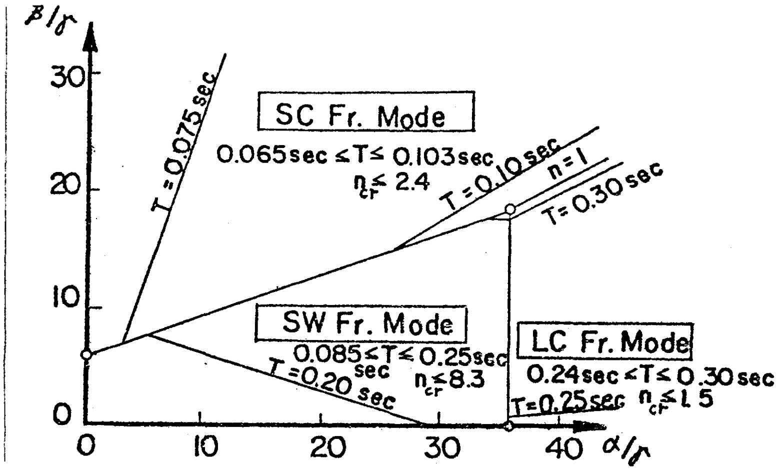
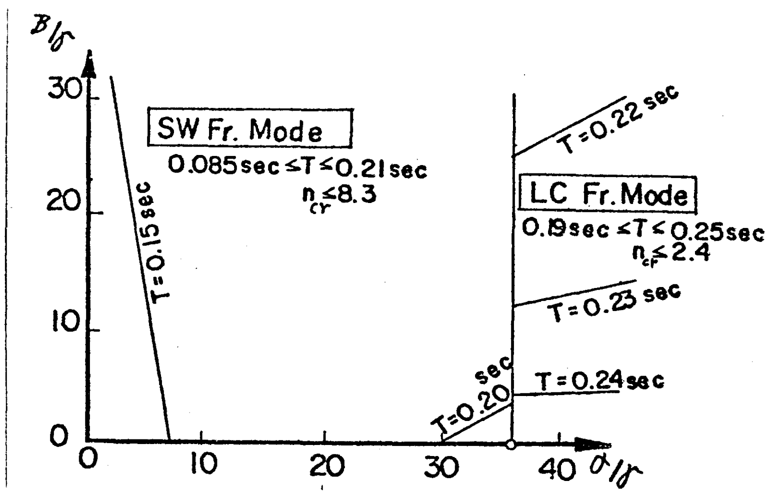


Fig.5 Wall Ratios of Reinforced Concrete Structures

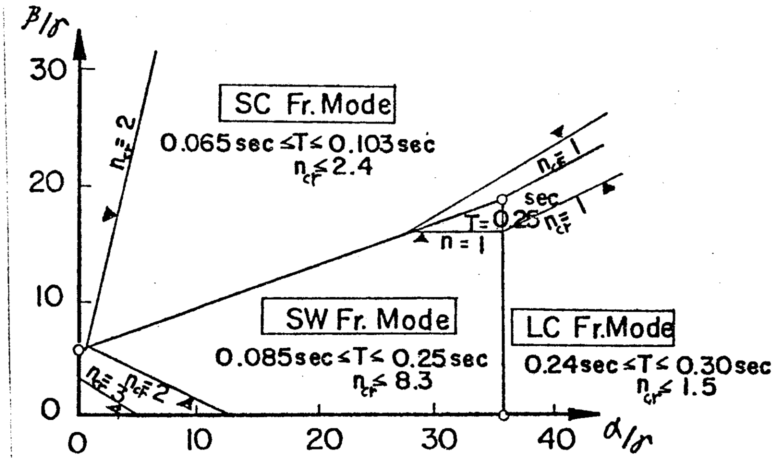


(a) (SCu with $p_h \fallingdotseq 0$ %)

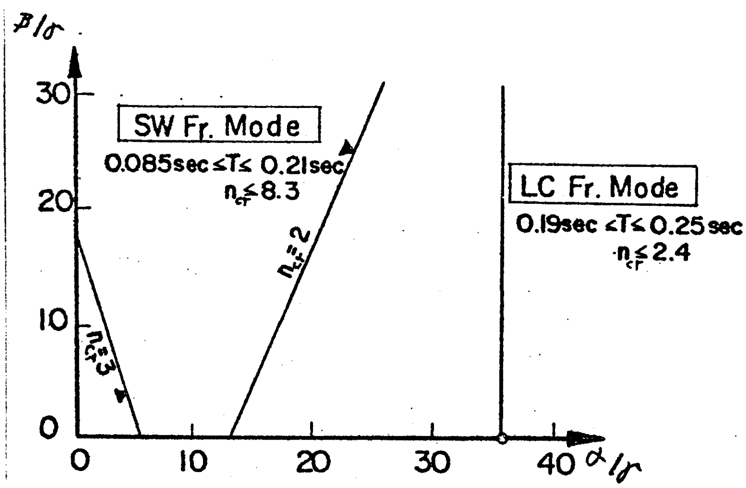


(b) (SCu with $p_h \geqq 1.0$ %)

Fig.6 Natural Periods of Reinforced Concrete Structures

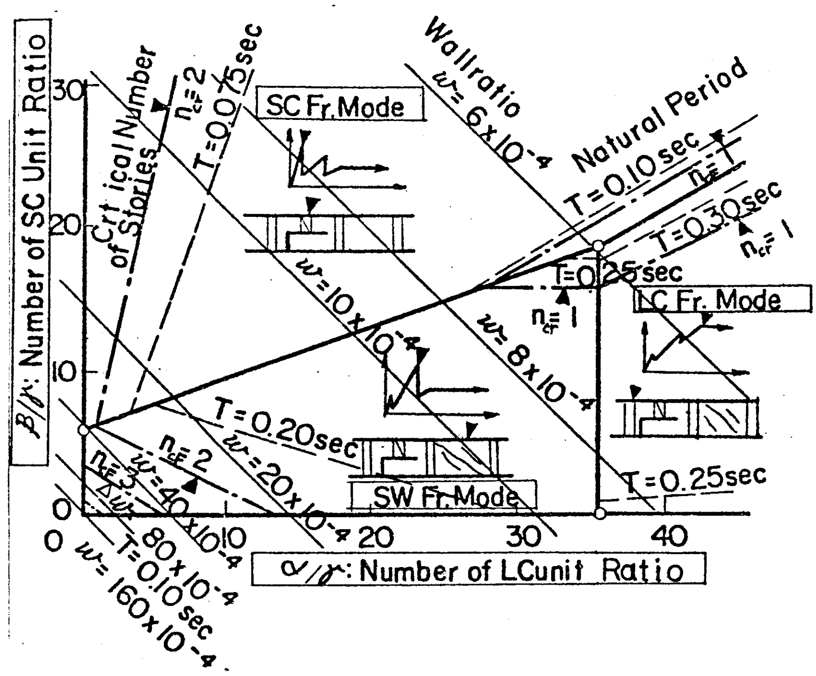


(a) (SCu with p_h ≒ 0 %)

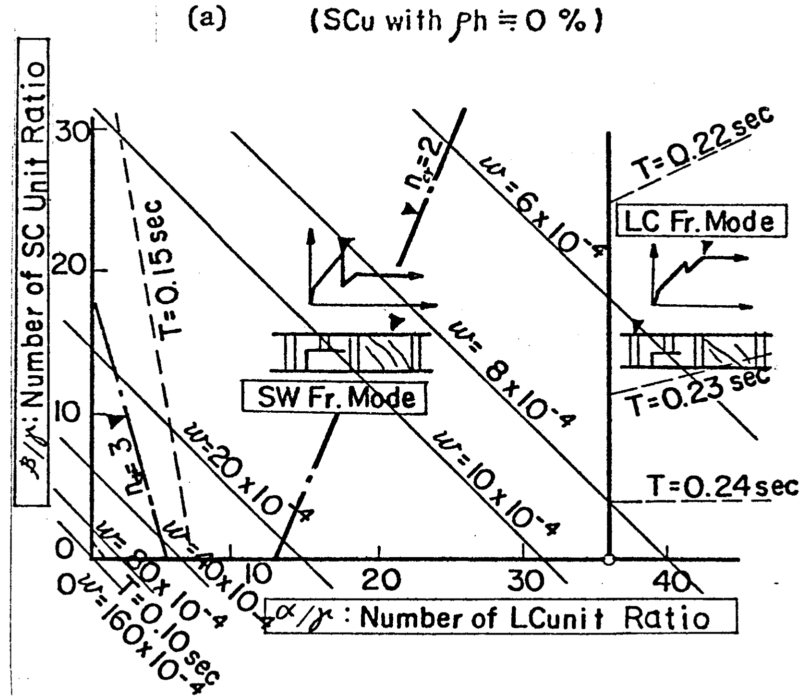


(b) (SCu with p_h ≥ 1.0 %)

Fig.7 Critical Number of Storys of Reinforced Concrete Structures



(a) (SCu with ρh ≒ 0 %)



(b) (SCu with ρh ≧ 1.0%)

Fig.8 Fracture Modes, Wall Ratios, Natural Periods and Critical Number of Stories

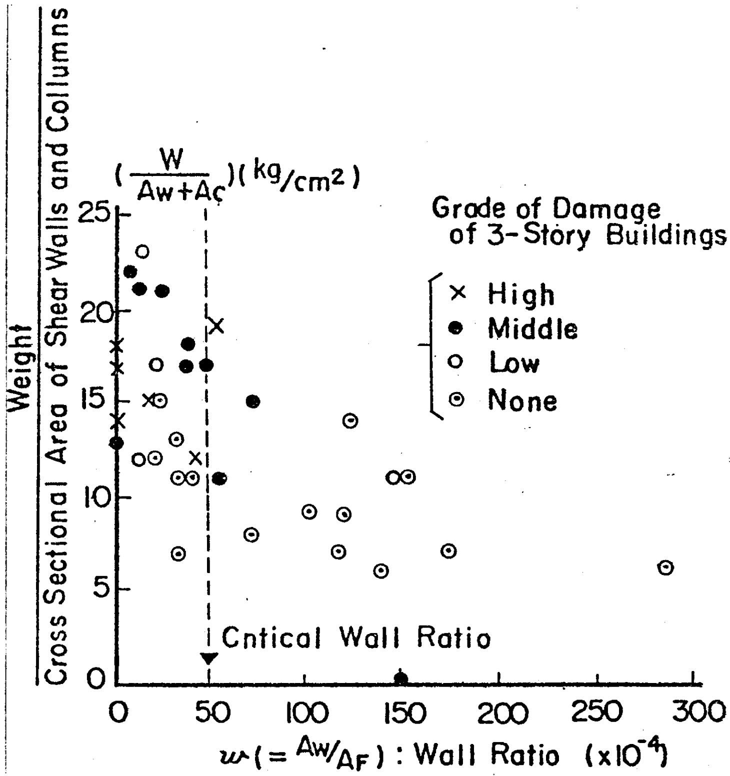
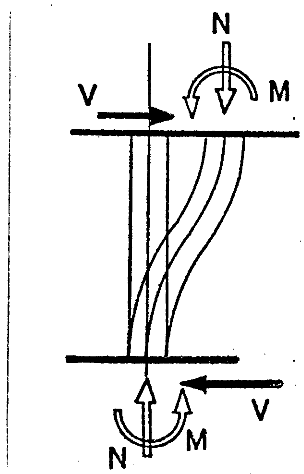


Fig. 9 Damage Statistics At Tokachi- Oki-Earthquake
(by Shiga,Shibata and Takahashi [1])



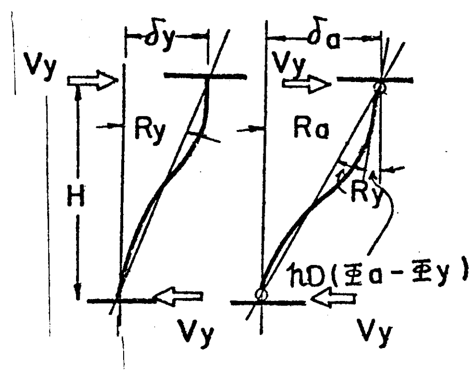


Fig. 10

Deformation Condition of Long Columns

Fig.11

Deformation of Long Columns

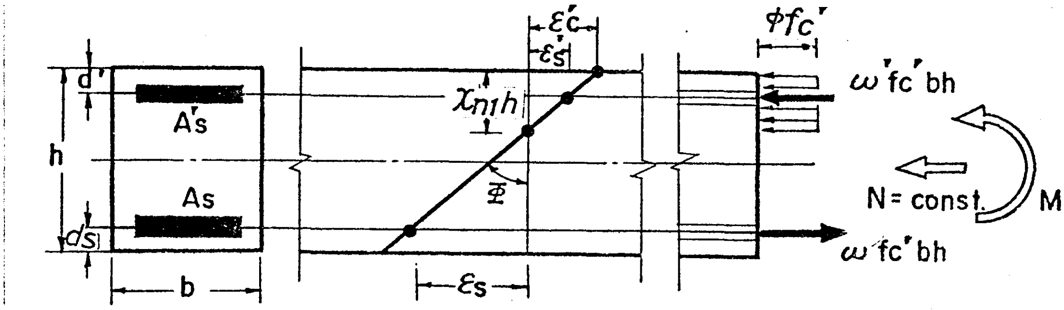


Fig. 12 Reinforced Concrete Cross Section, Strain and Stress

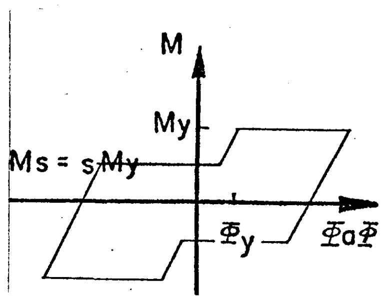


Fig.13 Moment-Curvature Hysteresis Loop

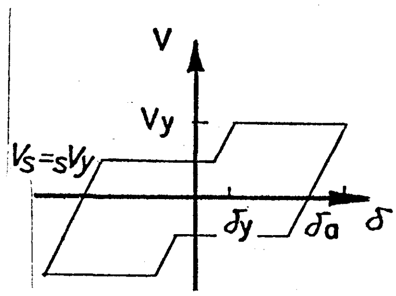


Fig.14 Story Shear Force-Displacement Hysteresis Loop

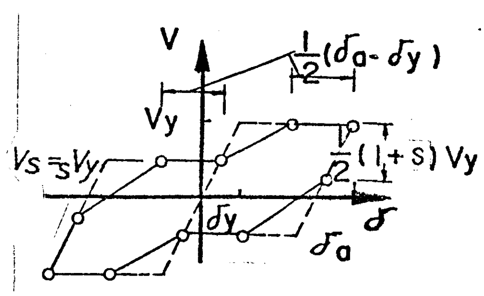


Fig.15 Story Shear Force-Displacement Hysteresis Loop

(with Bauschinger Effect)

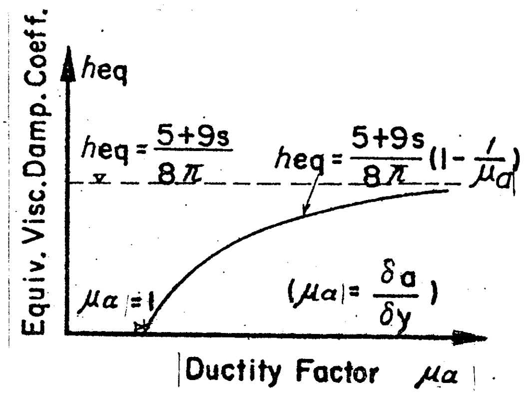


Fig.16 Equivalent Viscous Damping Coefficient - Ductility Factor Relationship

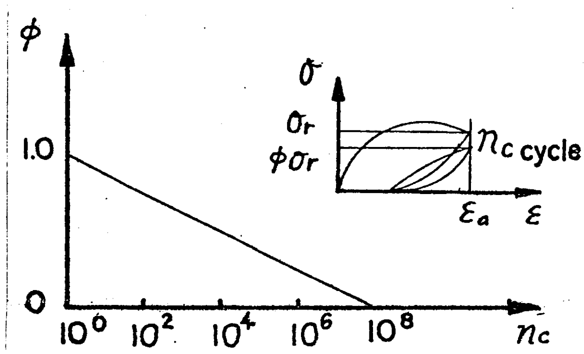


Fig.17 Deterioration Factor - Number of Loading Cycles of Plain Concrete

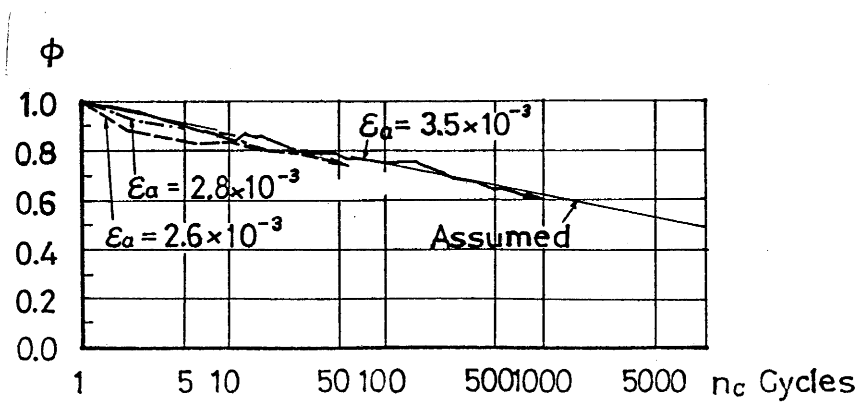


Fig.18 Deterioration Factor - Number of Loading Cycles of Plain Concrete (Experimental Data by Yamada and Shimada [3])

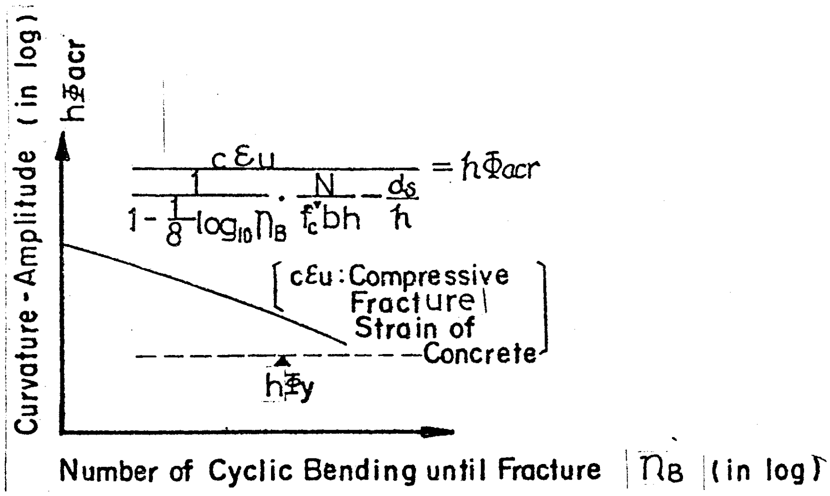


Fig.19 Critical Curvature Amplitude -Number of Cycles to Fracture

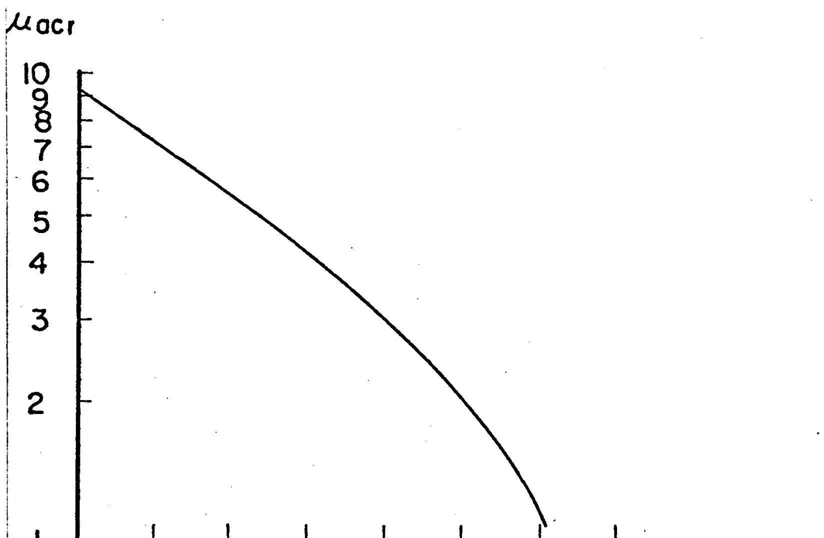


Fig.22 Critical Ductility Factor Amplitude - Number of Cycles to Fracture Relationship of Assumed Column

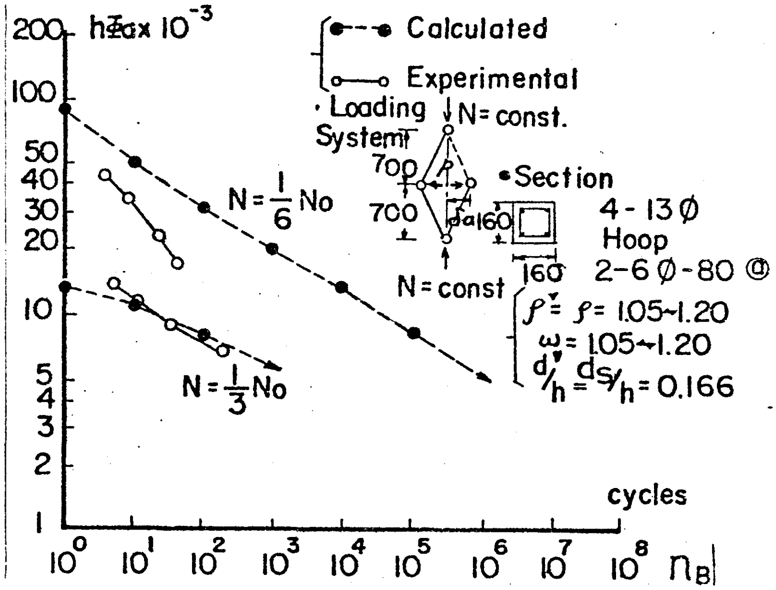
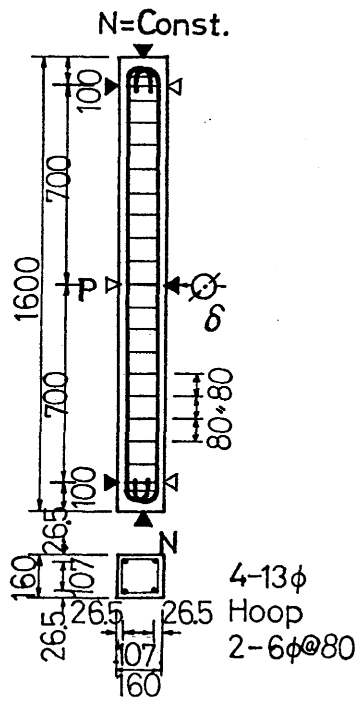
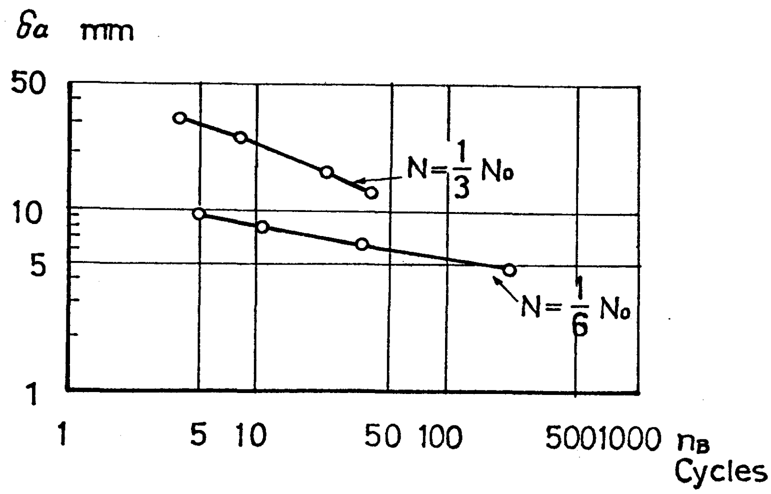


Fig.20 Curvature Amplitude-Number of Cycles to Fracture
(Tests and Analysis)



(a) Test Specimen



(b) Deflection Amplitude - Number of Cycles to Fracture (Experimental Data)

Fig.21 Low Cycle Fatigue Test of Long Colunns [4]

CHAPTER 5

REINFORCED CONCRETE STRUCTURES WITH ASYMMETRIC SHEAR WALLS

5-1. Introduction

In Chapter 3, the fundamental concept of aseismic capacity that reinforced concrete shear walls resist against earthquake loading through their shear resisting capacity and long columns through their hysteretic damping capacity was derived from the aseismic characteristics of reinforced concrete structures and members. In Chapter 4, this basic idea was applied to the real types of reinforced concrete structures, i.e., medium or low rise buildings with and without shear walls in order to estimate the aseismic capacity and safety of them.

In this chapter, furthermore, such evaluation methods of aseismic capacity are applied to two types of reinforced concrete structures, i.e., low rise buildings with shear walls asymmetrically arranged in plan. The aseismic capacity of this type is estimated on the basis of the combination of the aseismic concepts of rigid and flexible structures. When the shear rigidity and capacity of asymmetric shear walls are sufficient, the structures resist against earthquakes through the story shear resisting capacity of the shear walls at the center of torsional vibration and through the hysteretic damping capacity of flexural members, i.e., long columns.

After the asymmetric shear wall collapses with shear fracture modes, the resistance of it should be neglected in the process of the evaluation of aseismic capacity.

5-2. Assumed Conditions

In order to simplify the estimation method of the aseismic capacity and safety of reinforced concrete structures with asymmetrically arranged shear walls in plan, the following idealized conditions are assumed:

Assumed structure-- Reinforced concrete structure is assumed to be composed of rigid frames with one-story and rectangular plans, which contain only long columns and shear walls as aseismic elements. Long columns exist on grid points and shear walls on grid lines asymmetrically in plan. A typical plan and its notations are shown in Fig. 1, where G is the center of gravity, E is the center of rigidity, c and w indicate the elements of long column and shear wall.

Critical states-- When such structures are subjected to sinusoidal ground accelerations, they may reach the steady-state resonance of predominant torsional vibration. According to the fundamental concepts described in Chapter 3, such a state is able to be considered as the critical state in which the aseismic capacity of the structures is evaluated. In such a state, the followings are assumed as for structural members:

(1) The lateral displacement amplitudes of shear walls are under the critical ones at which the shear walls show the maximum shear resisting capacity.

(2) The lateral displacement amplitudes of long columns exceed yielding points.

(3) The long columns with the maximum excentricity from the center of torsional vibration have the critical displacement (or ductility) amplitudes.

Assumptions for analytical expressions--In order to express the critical states and aseismic capacity of the structures analytically, the followings are assumed:

(1) The loof slabs of the structures are rigid enough.

(2) The sinusoidal lateral forces act on the center of gravity.

(3) The structures show only torsional vibration around the center of rigidity, which is assumed to coincide with the center of torsional vibration.

(4) The center of rigidity resists against forced vibrations through the shear resisting capacity of shear walls under the condition that they have sufficient shear rigidity and capacity.

(5) The center of rigidity does not move during vibrations.

(6) The torsional rigidity of shear walls is neglected.

(7) The effects of viscous damping are neglected and only the hysteretic damping capacity of long columns is taken into account.

(8) In order to apply quantitatively the procedures mentioned above to real structures, the load-displacement characteristics of shear walls and long columns must be given as follows:

Shear walls--The load-displacement relationship of shear walls under one way monotonic loading is shown in Fig. 2, and V_{wy} and δ_{wy} are approximately given as follows (1):

$$V_{wy} = \frac{1}{4} f'_c A_w, \tag{1-a}$$

$$\delta_{wy} = 4x10^{-3} \cdot H. \tag{1-b}$$

Long columns--As for long columns subjected to cyclic bending in the direction of the principal axis of column-section, the moment-curvature hysteresis loop is the basic characteristic and shown in Fig. 3, where the characteristic values are expressed as follows:

$$M_y = \{\frac{1}{2}(1-x_{n1})x_{n1} + \omega\gamma\} f'_c bh^2, \tag{2-a}$$

$$M_s = \{1-(x_{n1}\frac{1}{\omega} -1)\}\omega \frac{1}{2}\gamma f'_c bh^2 , \tag{2-b}$$

$$\Phi_y = \frac{2\varepsilon_{sy}}{h\gamma}, \quad \text{(2-c)} \qquad \text{where} \quad x_{n1} = \frac{N}{f'_c bh} . \quad \text{(3)}$$

Based upon the moment-curvature hysteresis loop and the deterioration of rigidity of reinforcement due to the Bauschinger effect, the hysteresis loop of shear force V_c and relative story displacement of long columns is illustrated as shown in Fig. 4, where the characteristic values are expressed as follows:

$$V_{cy} = \frac{2M_y}{H}, \quad V_{sy} = \frac{2M_s}{H}, \quad \delta_{cy} = \frac{1}{6} \Phi_y H^2. \tag{4}$$

The relationship of torsional moment M_c and torsional angle θ of long columns around the torsional center of the structures is able to be shown in Fig. 5, where

$$M_{cy} = V_{cy} e_c, \qquad \theta = \frac{\delta_c}{e_c} . \tag{5}$$

(9) It must be noted that the long columns show flexural yielding type under the condition that

$$\overline{H} = \overline{H}_{cr} = \frac{\{X+2(1+X)\omega\}\gamma}{\frac{7}{8}\cdot\frac{d}{h}F(X)} , \tag{6}$$

$$\text{where} \quad F(X) = \sqrt{-0.1X^2 + 0.09X + 0.01} . \tag{7}$$

(10) The yielding condition of the long columns subjected to biaxial bending is given by circular function such as shown in Fig. 6 (2).

Dynamic Idealization of Structures

According to the assumptions and Fig. 1, the structures with asymmetric shear walls are able to be idealized as shown in Fig. 7.

The differential equation of motion for the dynamic model shown in Fig. 7 is expressed by

$$I_E\ddot{\theta} + \sum_c k_{\theta c} f_c(\omega t, \theta) + (\sum_w e_w^2 k_w)\theta = -me_G \alpha_0 \cos(\omega_0 t + \psi) , \tag{8}$$

where I_E is the polar moment of inertia of the mass around E, which is given by

$$I_E = me_G^2 + I_G, \tag{9}$$

$k_{\theta c}$ is the polar moment of k_c of long column element c around E, which is given by

$$k_{\theta c} = e_c^2 k_c, \tag{10}$$

$f_c(\omega t, \theta)$ is the function of restoring force of long column element c shown in Fig. 5.

In the case of steady-state resonance, by integrating the both sides of Eq.8 with respect to θ over one cycle, the left side becomes

$$\oint \sum_c k_{\theta c} f_c(\omega t, \theta) d\theta, \text{ i.e., } \sum_c A_c,$$

where A_c is the area of the $M_c - \theta$ hysteresis loop in Fig. 5, and the right side becomes

$$m e_G \alpha_0 \theta_a \pi \text{ , because } \psi = \sin^{-1}(-1) = -\frac{\pi}{4} .$$

On the other hand, the equivalent viscous damping coefficient of the $M_c - \theta$ hysteresis loop h_{eq} is expressed by

$$h_{eq} = \frac{1}{4\pi} \frac{A_c}{\frac{1}{2} M_{cy} \theta_a} . \tag{11}$$

Therefore, the left side of Eq. 8 becomes $2\pi\theta_a \Sigma(h_{eq} M_{cy})$.

Finally, as the equiribrium equation of energy in the steady-state resonance of torsional vibration, the following expression is derived:

$$2\pi\theta_a \Sigma(h_{eq} M_{cy}) = m e_G \alpha_o \theta_a \pi , \tag{12}$$

then

$$2 \Sigma(h_{eq} M_{cy}) = m \alpha_o e_G. \tag{13}$$

5-3. Aseismic Criterion Equations

Criterion Equations of Aseismic Safety

It is reasonable that Eq. 13 is similar to the criterion equation of aseismic safety of long columns, $m\alpha_o = 2h_{eq}V_a$, which is derived in

Chapter 3. Consequently, Eq. 13 is the criterion equation of the aseismic safety of the reinforced concrete structures with asymmetric shear walls, and then the right side of it, i.e., RESONANCE CAPACITY corresponds to the aseismic capacity of them. In addition, according to the assumptions that (1) the lateral displacement of shear walls are always below δ_{wy}, and (2) the center of rigidity is subjected to the lateral load W, the following criterion equations of shear walls should be satisfied:

$$\text{in x-direction;} \quad \frac{W}{\sum_{w} k_{wx} + \sum_{c} k_{cx}} + \theta_{cr}(e_{wy})_{max} \leqq \delta_{wy} \quad , \tag{14}$$

$$\text{in y-direction;} \quad \frac{W}{\sum_{w} k_{wy} + \sum_{c} k_{cy}} + \theta_{cr}(e_{wx})_{max} \leqq \delta_{wy} \quad , \tag{15}$$

where

$$\theta_{cr} = \frac{\mu_{cr}\delta_{cy}}{(e_c)_{max}} \quad , \tag{16}$$

which is decided by the fatigue characteristics of the long columns with the maximum e_c.

Using the equation $h_{eq} = \frac{5+9s}{8\pi}(1-\frac{1}{\mu_{cr}})$ for the slipping type of hysteresis loop derived in Chapter 4, Eq. 13 becomes

$$\sum_{c}[\frac{5+9s}{4\pi}(1-\frac{1}{\mu_{crc}})V_{cy}(e_c)] = m\,\alpha_0 e_G \quad , \tag{17}$$

where

$$\mu_{cr} = \theta_{cr}/\theta_{cy} = (\delta_{ca}/e_c)/(\delta_{cy}/e_c) = \delta_{ca}/\delta_{cy}, \tag{18}$$

and δ_{ca} and δ_{cy} are the critical deflection amplitude and yielding deflection of the long columns with max. e_c. Of course, in the case that $\mu_{cr} \leqq 1$, h_{eqc} becomes zero.

Fatigue Characteristics

On the application of Eq. 17 to real cases, the critical ductility amplitude μ_{cr} of the long columns with max.e_c must be given. Regarding reinforced concrete square cross section subjected to uniaxial cyclic bending moments (see Fig. 8), the relationship between curvature amplitude and the number of cycles until fracture was already derived in Chapter 4 as follows:

$$\frac{{}_c\varepsilon_u}{\dfrac{1}{1-(1/8)\log_{10}n_B}\cdot\dfrac{XN_o}{f'_c bh}-\dfrac{d'}{h}} = h\Phi_{acr}. \tag{19}$$

Then the critical ductility amplitude μ_{cr} was expressed by

$$\mu_{cr} = 1 + \frac{h(\Phi_a - \Phi_y)H}{\delta_{cy}}. \tag{20}$$

On the other hand, the long columns in such structures as show torsional vibration are subjected to bi-axial cyclic bending moments. As an extreme case, let a square cross section be undergo diagonal bending moment such as shown in Fig. 9. If the same assumptions for fatigue collapse as those in Chapter 4 are applied, axial force N is given by

$$N = XN_o = (x_{nl}h)^2 f'_c \left(1-\frac{1}{8}\log_{10}n_B\right), \tag{21}$$

and curvature amplitude is given by

$$h\Phi_a = \frac{{}_c\varepsilon_u}{x_{nl}-\sqrt{2}(d'/h)}. \tag{22}$$

Consequently, $h\Phi_a$-n_B relationship for diagonal bending, the following equation is given:

$$\frac{{}_c\varepsilon_u}{\sqrt{\dfrac{XN_o}{f_c'bh(1-\frac{1}{8}\log_{10}n_B)}}-\sqrt{2}\,\dfrac{d'}{h}} = h\Phi_a. \tag{23}$$

Comparing $h\Phi_a$ in Eq. 19 with that in Eq. 23 under the conditions that $XN_o/f_c'bh = 1/6$, $d'/h = 0.1$, $n_B = 200$ cycles, the deterioration ratio becomes

$$\frac{\dfrac{XN_o}{f_c'bh(1-1/8\log_{10}n_B)}-\dfrac{d'}{h}}{\sqrt{\dfrac{XN_o}{f_c'bh(1-1/8\log_{10}n_B)}}-\sqrt{2}\,\dfrac{d'}{h}} = 0.367. \tag{24}$$

Judging from the value of such a deterioration ratio, in order to simplify the analytical expression, here, the critical ductility amplitude of square cross section under bi-axial cyclic bending is proposed to be:

$$\mu_{cr\theta} = \mu_{cr}\frac{1}{\sin\theta + \cos\theta}, \tag{25}$$

such as shown in Fig. 10 as a diamond figure.

5-4. Application to Real Structures With Asymmetric Shear Walls

Outlines of Assumed Structure

At the Tokachi-Oki Earthquake in 1968, the Hachinohe City Library, a one-story reinforced concrete structure with extremely asymmetric shear walls, was damaged due to torsional deformations (3). As an example, the aseismic capacity and safety of a reinforced concrete structure, which is similar to the Hachinohe City Library and is simplified more than the proto type, are investigated. The outlines of the assumed structure are as follows:

(1) The plan and section of it are shown in Fig. 11.

(2) The sections of beams, columns and shear walls are shown in Fig. 12. The thickness of slabs is 13.5cm (5.3in).

(3) Concrete strength for design is f_c'=180kg/cm^2(2560psi), and yielding strength of reinforcement for design is f_y=2400kg/cm^2(34.1ksi).

(4) Considering the effects of rigid pannel action of slabs, the parts of rigid frames without shear walls show column-yielding type.

Judging from the proportion of the plan, the torsional vibration due to horizontal force in x-direction is considered to be predominant, which coincides with the real fracture mode of the Hachinohe City Library, so that, here, the aseismic capacity of this kind of deformation is discussed.

Load-Displacement Relationships of Shear Walls

As for the structure shown in Fig. 11, A_w is given as follows:

for (5) axis; A_w = 12x(720x3-240-200) = 20600 cm^2,

for (5') axis; A_w = 12x(720x2-100) = 16100 cm^2.

Then V_{wy} is given as follows:

for (5) axis; $V_{wy} = (1/4)x180x20600x10^{-3} = 930$ tons,

for (5') axis; $V_{wy} = (1/4)x180x16100x10^{-3} = 720$ tons.

The critical displacement δ_{wy} is given by

$$\delta_{wy} = 4x10^{-3}x420 = 1.68cm.$$

Consequently, the load displacement characteristics of the shear walls subjected to monotonic loading on (5) and (5') axes in x-direction are shown in Fig. 13.

Load-Displacement Relationships of Long Columns

The characteristic values of the hysteresis loop of long column-elements (including beam-elements) are given in accordance with flexural equations described above as follows:

$$V_{cy} = 9.0 \text{ tons}, \qquad \delta_{cy} = 2.5 \text{ cm}, \qquad s = 0.2 .$$

And μ_{cr} is assumed to be 4.0 because of relatively small axial forces. The hysteresis loop and skeleton curve are shown in Fig. 14

The Center of Torsional Vibration

As the first approximation, the center of rigidity in the x-direction assumed to be determined only by the rigidity of shear walls and to be located on the position with 1.6m (=3.6mx430/980) distance from (5) axis to the right hand side (see Fig. 15). As the next step, the position of the center of torsional vibration is calculated by means of the corrected rigidity in the x-direction shown in Fig. 15, where the torsional deformation of the structure is shown around the firstly assumed center of rigidity, which is limited by the critical ductility, 4.0, of the columns with max.e_c.

Finally, the position of the center of rigidity, which is assumed to coincide with the center of torsional vibration, is given by the distance from (1) axis to the right hand side,

$$y_E = \frac{0.9\times4\times0+1.2\times4\times7.2+1.7\times4\times14.4+3.1\times4\times21.6+3.6\times4\times36.0+ +550\times28.8+430\times32.4}{0.9\times4+1.2\times4+1.7\times4+3.1\times4+3.6\times4+550+430} = 30.03 \text{ m.}$$

Examination of Shear Walls Deformation

Let the seismic weight of the structure be about 1000 tons (2200 kips). On the other hand, the shear resisting capacity of the all shear walls in the x-direction is 1650 tons (=930t+720t, see Fig. 13). Therefore, the center of torsional vibration has sufficient static shear resisting capacity. If it has poor shear resisting capacity, the aseismic capacity of the structure should be estimated as a flexible type.

By using Eq. 14 , the examination of the deformation of (5') axis shear walls is able to be carried out as follows:

$$W = 1000\text{tons},\quad \Sigma k_{wx} = 550\text{t/cm}+430\text{t/cm (see Fig. 13)},$$

$$\sum_c k_{cx} = 0.9\times4+1.2\times4+1.7\times4+3.1\times4+3.6\times4 = 42.0 \text{ t/cm},$$

$$\theta_{cr} = 4\times2.5\text{cm}/3003\text{cm} = 0.00333 \text{ rad.},$$

$$(e_{wy})_{max} = 360\text{cm}-(3003\text{cm}-4\times720\text{cm}) = 257\text{cm, for (5') axis,}$$

then

$$\frac{W}{\sum_w k_{wx}+\sum_c k_{cx}} + \theta_{cr}(e_{wy})_{max} = 1.0 \text{ cm} \leqq \delta_{wy} = 1.68 \text{ cm.}$$

Consequently, it is found that (5') axis shear walls will never collapse before (1) axis columns fracture due to flexural fatigue.

Evaluation of Aseismic Capacity

In steady-state resonance of torsional vibration, the aseismic capacity is estimated by the left side of Eq. 17. Considering the lateral displacement in the x-direction, the critical ductility factor must be corrected as follows:

$$\frac{W}{\sum_w k_{wx} + \sum_c k_{cx}} = \frac{1000}{550+430+42.0} = 0.98 \text{ cm},$$

then $$\mu_{cr} = (4\text{x}2.5-0.98)/2.5 = 3.6 \rightarrow 3.5 .$$

Consequently, the aseismic capacity is calculated as follows:

$$\sum_c \left[\frac{5+9s}{4\pi}(1-\frac{1}{\mu_{crc}})V_{cy}(e_c)\right] = \frac{5 + 9\text{x}0.2}{4\pi}\text{x}9\text{x}[4\text{x}(30.03+22.83+$$

$$+15.63+8.43)-\frac{16\text{x}30.03}{3.5}] = 838 \text{ tm.} \quad (26)$$

The judgement of aseismic safety is performed on the assumption that $\alpha_o/g = 0.3$. The right side of Eq. 17 is given by

$$m\alpha_o e_G = 1000\text{x}0.3\text{x}(30.03-\frac{1}{2}\text{x}36.0) = 3609 \text{ tm.} \quad (27)$$

Comparing the value of Eq. 26 with that of Eq. 27, the aseismic safety of the assumed structure against torsional vibration proved not to be sufficient. In fact, the Hachinohe City Library was heavily damaged due to the torsional deformation around the asymmetric shear walls (3).

5-5. Concluding Remarks

The aseismic capacity of low rise building with asymmetric shear walls is investigated quantitatively, on the basis of the aseismic concepts described in Chapter 3. The analytical procedures and equations used for the evaluation of aseismic capacity and the concrete examples of application to real types of reinforced concrete structures are shown.

As for low rise reinforced concrete structures with asymmetric shear walls, the followings are concluded:

(1) When the shear capacity of asymmetric shear walls are enough to resist against the lateral load equal to seismic weight, the aseismic capacity of structures is able to be estimated on the condition of steady-state resonance of torsional vibration.

(2) The aseismic capacity of such structures is evaluated by the left side of Eq. 13 or Eq. 17, i.e., the hysteretic damping capacity of long column elements.

(3) The aseismic safety is also examined by the aseismic criterion equation, Eq. 13 or 17.

(4) As a result of application of Eq. 17 to a typical type of real reinforced concrete structure (see Figs. 11, 12), its aseismic safety proved not to be sufficient.

(5) It is shown the aseismic capacity of structures with asymmetric shear walls is proportional to the hysteretic damping capacity of the

columns and to their distances from the center of torsional vibration, but is reciprocal of the distance between the center of gravity and that of rigidity, e_G (see Eq. 17).

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(2) Yamada, M.; Konishi, T., "Elasto-plastic Biaxial Flexural Behavior of Reinforced Concrete Columns Subjected to Axial Loads, (IV. Analysis), Research Report of Kinki Branch of AIJ, May 1971, pp. 61-64, "(V. Summary, Test and Analysis)", Transactions, Extra, Annual Meeting, AIJ, Nov. 1971, pp. 763-764.

(3) Architectural Institute of Japan, "Report on the Investigation of Damage due to 1968 Tokachi-oki Earthquake, (in Japanese)," Dec. 1968, pp. 432-452.

NOTATION

A_w = cross sectional area of shear walls

A_c = area of $M_c - \theta$ hysteresis loop of long column element c

b = width of cross section of columns

c = notation of long column element

d = distance from extreme compression fiber to centroid of tension reinforcement

d' = distance from extreme compression fiber to centroid of compression reinforcement

d_s = distance from centroid of tension reinforcement to the tensile face of columns

E = center of rigidity of structures

e_c = distance from E to long column element c

$(e_c)_{max}$ = maximum value of e_c

e_G = distance from E to G

e_w = distance from E to shear wall element w

F( ) = fracture criterion equation of concrete under combined normal and shear stresses

f'_c = compressive strength of concrete

f_y = yield strength of reinforcement

$f_c(\omega t, \theta)$ = resisting force function of M_c

G = center of gravity of structures

g = acceleration of gravity

H = story height of long columns and shear walls

H^* = story height of short columns

H_{cr} = critical story height

$\overline{H}$ = H/h

$\overline{H}_{cr}$ = H_{cr}/h

h = total depth of cross section of columns

h_{eq} = equivalent damping coefficient

I_E = polar moment of inertia of mass around E

I_G = polar moment of inertia of mass around G

K = stiffeness of story of structures

$k_{\theta c}$ = polar moment of stiffeness k_c $(=e_c^2 \cdot k_c)$

k_c = stiffeness of long column element c

k_w = stiffeness of shear wall element w

M_c = torsional moment of long column element c around E $(=e_c \cdot V_c)$

M_{cs} = slipping M_c

M_s = slipping bending moment

M_y = yield bending moment

M_{cy} = yielding M_c

m = mass

N = axial force

N_o = ultimate strength of centrally loaded columns

n_c = number of cycles

R = relative story displacement angle

R_a = amplitude of R

R_y = yielding R

s = slipping coefficient $(=M_s/M_y,\ =M_{cs}/M_{cy})$

t = thickness of shear walls, time

V = relative story shear force

V_y = yielding V

V_{cy} = V_y of long column element c

V_c = V of long column element c

V_{cs} = slipping V_c

V_w = V of shear wall element w

V_{wy} = V_y of shear wall element w

W = seismic weight of structures

w = notation of shear wall element

x = co-ordinate, suffix of direction

x_{nl} = ratio of distance from extreme compressive fiber to neutral axis to the total depth of cross section of columns

y = co-ordinate, suffix of direction

y_E = distance from end axis to E of structure

$\ddot{z}_G$ = acceleration of ground motion

α_o = acceleration amplitude of sinusoidal ground motion

γ = ratio of distance center-to-center of reinforcement on opposite faces of columns to h

δ = relative story displacement

δ_c = δ of long column element c

δ_{ca} = amplitude of δ_c

δ_{cy} = yielding δ_c

δ_w = δ of shear wall element w

δ_{wy} = fracture δ_w

${}_s\varepsilon_y$ = yield strain of reinforcement

${}_c\varepsilon_y$ = (idealized) yield compressive strain of concrete

${}_c\varepsilon_u$ = ultimate compressive strain of concrete

Φ_a = curvature amplitude

Φ_{acr} = critical Φ_a

Φ_y = yielding curvature

θ = torsional angle of structure

θ_{cr} = critical torsional angle

θ_a = torsional angle amplitude $(= \delta_{ca}/e_c)$

μ_{crc} = ductility factor of long column element c at θ_{cr}

μ_{cr} = critical ductility factor

$\mu_{cr\theta}$ = μ_{cr} of cross section under θ-direction cyclic bending

χ = $({}_s\varepsilon_y/{}_c\varepsilon_y)$

ω = tensile reinforcing index, natural circular frequency

ω' = compressive reinforcing index

ω_0 = circular frequency of earthquake waves

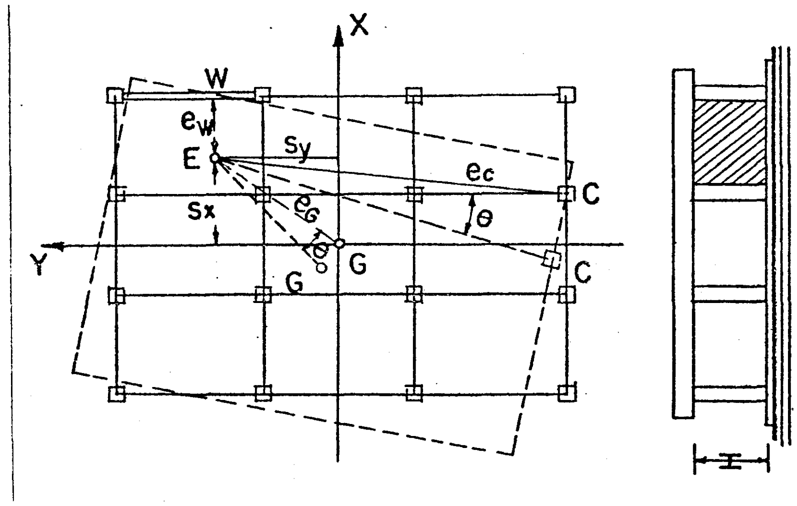


Fig. 1 Plan and Notations of Structures With Asymmetric Shear Walls

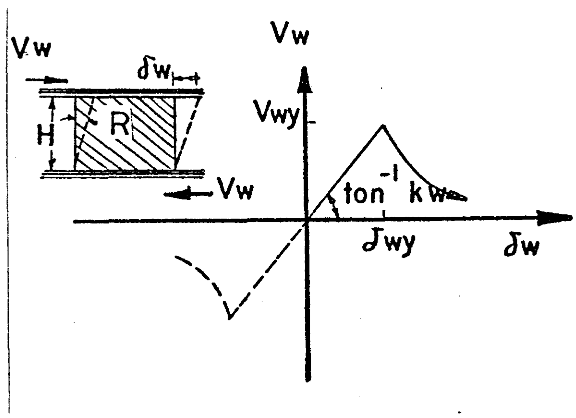


Fig. 2 Story Shear Force - Displacement Relationship of Shear Wall

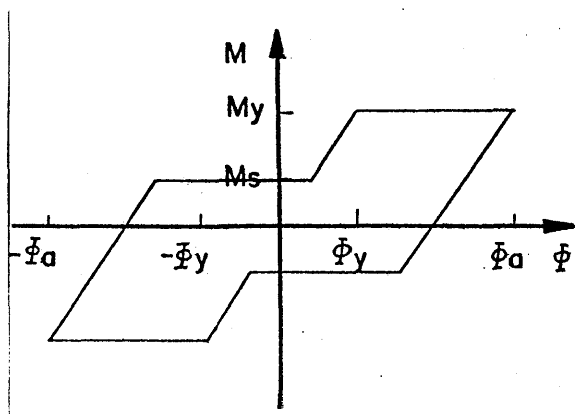


Fig. 3 Moment-Curvature Hysteresis Loop of Long Column Element

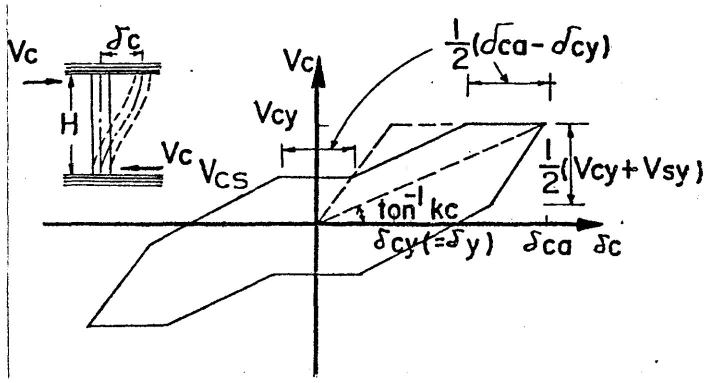


Fig. 4 Story Shear Force - Displacement Relationship of Long Column Element

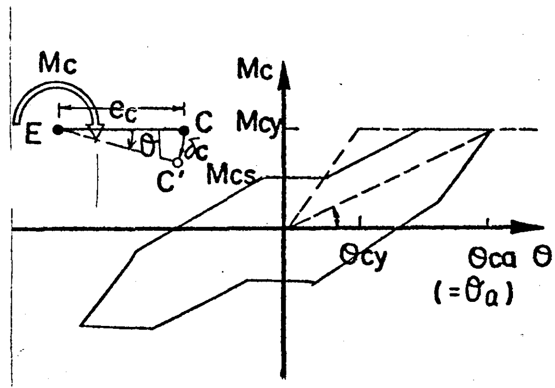
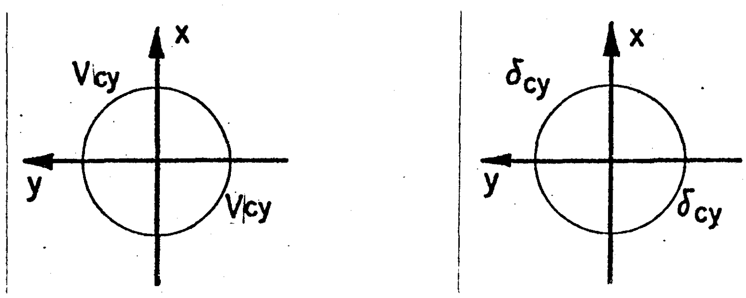


Fig. 5 Torsional Moment - Angle Hysteresis Loop of Long Column Elements



(a) Yield Story Shear Force (b) Yield Displacement

Fig. 6 Yield Function of Long Column Elements

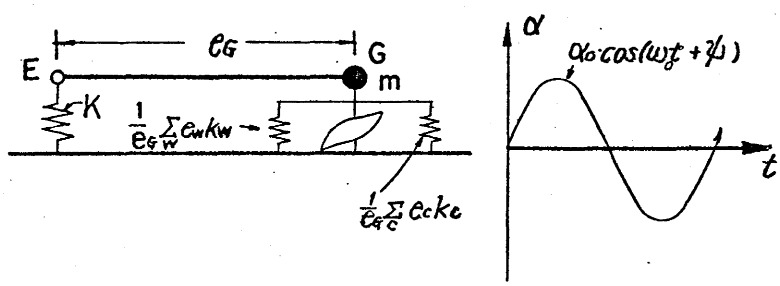


Fig. 7 Dynamic Model of Structure With Asymmetric Shear Wall

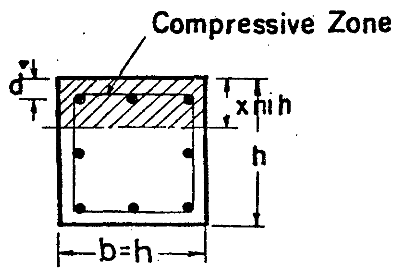


Fig.8 Cross Section, Bent around Principal Axis

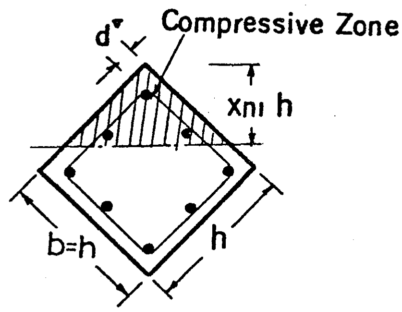


Fig.9 Cross Section, Bent around Diagonal Axis

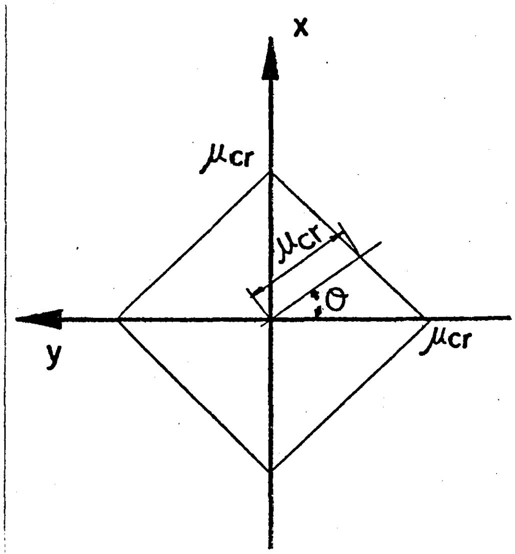


Fig.10 Critical Ductility Factor Function of Columns under Biaxial Bending

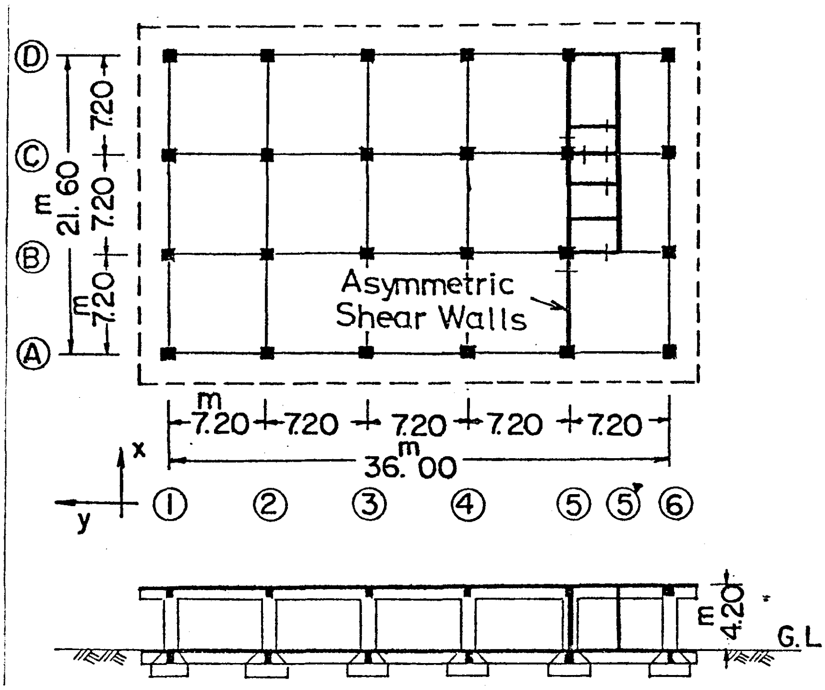
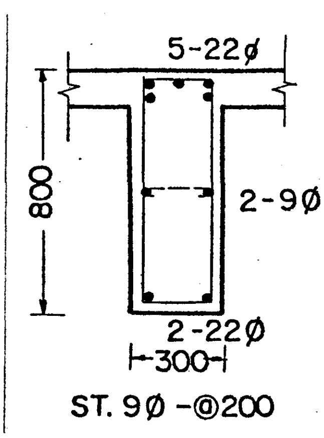
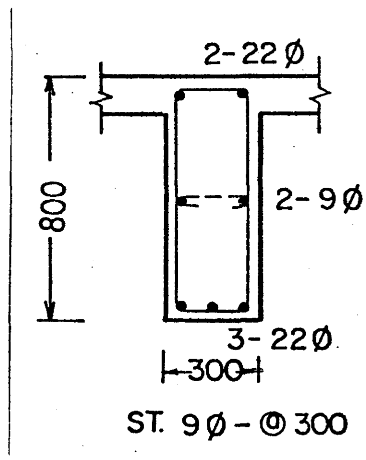


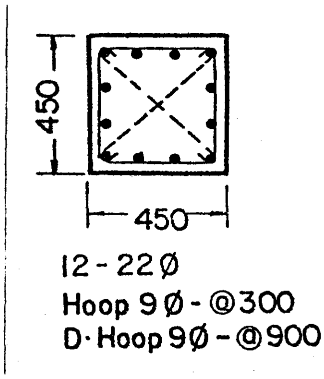
Fig.11 Plan and Section of Assumed Structure
With Asymmetric Shear Walls



(b) Span-End of Beam



(c) Span-Center of Beam



(a) Column

Fig.12 Assumed Cross Section
of Beams and Columns

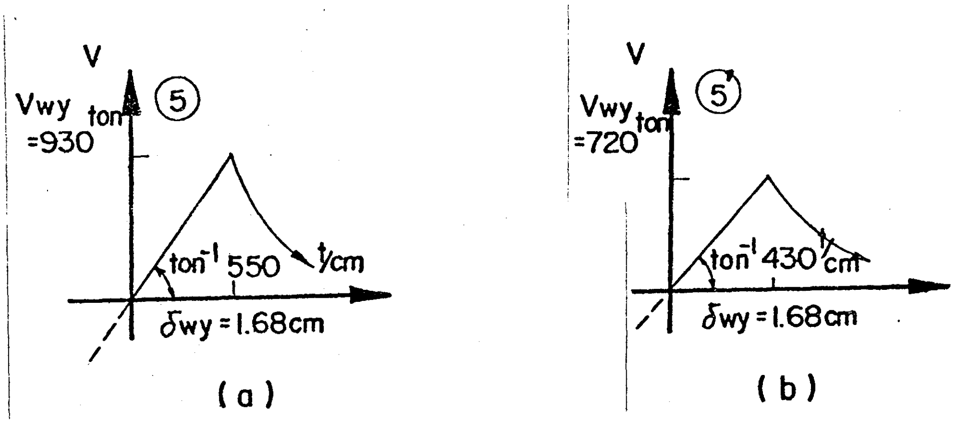


Fig.13 Story Shear Force-Displacement Relationships of Assumed Shear Walls

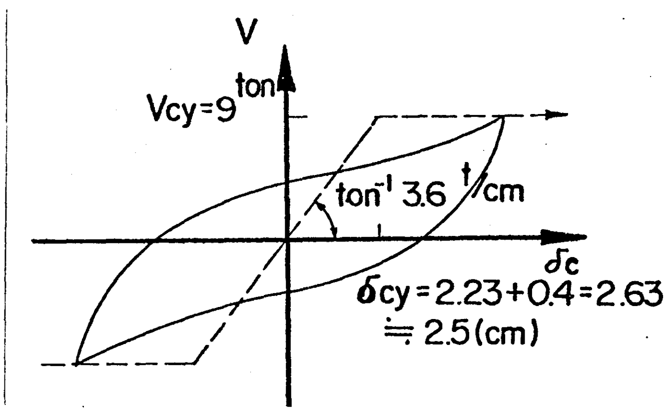


Fig.14 Story Shear Force-Displacement Hysteresis Loop of Assumed Long Columns

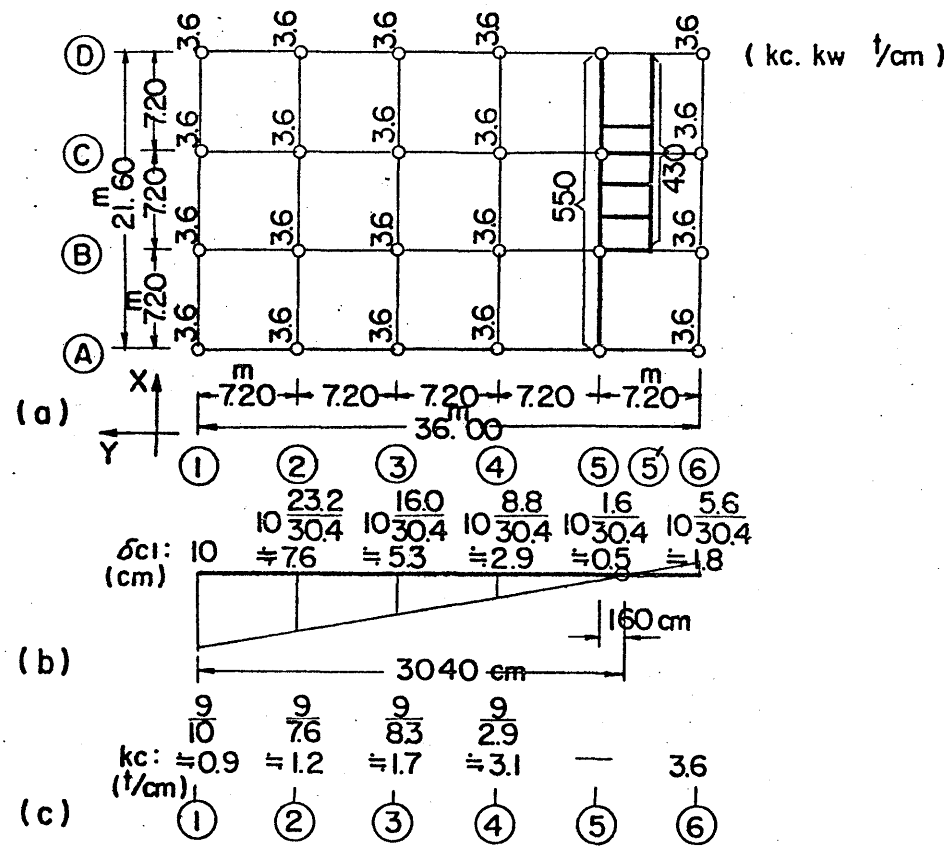


Fig.15 Stiffeness Distributions Before and After Rotation of Assumed Structures

CHAPTER 6

MULTI-STORY REINFORCED CONCRETE STRUCTURES WITH AND WITHOUT CANTILEVER-TYPE SHEAR WALLS

6-1. Introduction

In this Chapter, the evaluation method of aseismic capacity are applied to multi-story reinforced concrete buildings with and without cantilever-type shear walls. The aseismic capacity of this type building is evaluated by the same way as applied to the flexible structures described in Chapter 4. If the cantilever-type shear walls in multi-story structures never collapse due to shear force and bending moment, which should be aimed in design, such structures belong to flexible type and the yielding phenomenon of it is caused by the flexural yielding of beams and columns in the part of rigid frames and of beams adjacent to shear walls. Therefore, this type of reinforced concrete structures also resists against earthquakes through the hysteretic damping capacity.

After the shear walls collapse with shear fracture mode, the resistance of shear walls should be neglected in the farther processes of the evaluation of aseismic capacity.

6-2. Assumed Conditions

The fundamental concept of aseismic capacity described in Chapter 3 is able to be applied to multi-story reinforced concrete structures with shear walls. The basic idea applied here, is that only the hysteretic damping capacity of the plastic hinges of flexural members in structures resists against earthquakes under the condition that shear walls never collapse. The equations used in the estimation procedures for aseismic capacity are derived from the following assumptions on multi-story structures:

(1) Multi-story reinforced concrete structures consist of rigid frames including cantilever type shear walls (see Fig. 1-a).

(2) The yielding of the structures subjected to lateral loads is caused by the plastic hinges which occurs at the ends of beams and columns in frames and at the ends of beams adjacent to shear walls.

(3) Cantilever type shear walls are designed neither to show flexural nor shear yielding before the yielding of beams and columns.

(4) As simulated and simplified waves of earthquakes, sinusoidal ground acceleration $\ddot{z}_G=\alpha_o\cos\omega_0 t$ is considered.

(5) The aseismic capacity of the structures is determined under the condition of steady-state resonance, and input energy is absorbed only by the hysteretic damping energy of flexural yielding beams and columns.

6-3. Aseismic Criterion Equations

Criterion Equations of Aseismic Safety

A typical type of multi-story reinforced concrete structures with cantilever type shear walls is shown in Fig. 1 -a, and its mechanically idealized model is shown in Fig. 1-b(1). There exists the following relationship between the vector matrix $\{R\}$ of relative story displacement angle and $\{V\}$ of story shear force:

$$\left[[K_F] + [K_B] + [K_W]\right] \{R\} = \{V\} , \qquad (1)$$

where $[K_F]$, $[K_B]$ and $[K_W]$ represent the stiffeness matrices of rigid frames without shear walls, beams adjacent to shear walls and shear walls, respectively.

According to the assumption (4), the right side of Eq. 1 is expressed by

$$\{V\} + [M]\{\ddot{Y}\} = -[M] \{1\} \ddot{z}_G = -[M]\{1\} \alpha_o \cos\omega_o t , \qquad (2)$$

where $[M]$ is the mass matrix of the structure. By integrating with respect to $\{R\}$ over one cycle, Eq. 2 is expressed by

$$\oint \left\{\left[[K_F] + [K_B] + [K_W]\right]\{R\} + [M] \{\ddot{Y}\}\right\} \cdot d\{R\}^T = -[M]\{1\}\alpha_o \oint \cos\omega_o t \cdot d\{R\}^T . \quad (3)$$

Assuming that $\{R\} = \{R_a \cos(\omega_o t + \psi)\}$, (4)

and according to the assumptions (1)(2)(3)(5), Eq. 3 finally becomes

$$A_{Fi} + A_{Bi} = \left(\sum_{j=i}^{n} W_j\right) \frac{\alpha_o}{g} \pi R_{ai} , \quad (i=1,\cdots,n), \qquad (5)$$

where W_j is the weight of j_{th} story, and A_{Fi} and A_{Bi} are the area of V-R hysteresis loops of rigid frames and beams adjacent to shear walls in i_{th} story, respectively.

As an aseismic criterion equation, finally, the following expression is given:

$$2\Sigma(V_{yi}h_{eqi}) = (\sum_{j=i}^{n} W_j)\frac{\alpha_o}{g}, \tag{6}$$

where V_{yi} and h_{eqi} are the yielding story shear force and the equivalent viscous damping coefficient of V-R hysteresis loops of columns and beams in rigid frames and beams adjacent to shear walls in i_{th} story. In order to use Eq. 6 as an aseismic criterion equation quantitatively, the following aseismic characteristics are necessary.

V-R hysteresis loop--The moment M - curvature Φ hysteresis loop of reinforced concrete cross section was already given by Eq. 2 in Chapter 5 (see Fig. 13 in Chapter 4). The hysteresis loops of story shear force V and relative story displacement angle R of beams and columns are shown in Fig. 2, where V_y, V_s, R_y are determined according to the deflection and yielding types of members such as shown in Fig. 3.

For column yielding type (see Fig. 3-a);

$$V_y = 2M_y/H, \; V_s = 2M_s/H, \; R_y = 1/6\cdot\Phi_y H. \tag{7}$$

For beam yielding type (see Fig. 3-b);

$$V_y = 2M_y/L\cdot L/H^*, \; V_s = 2M_s/H^*, \; R_y = 1/6\cdot\Phi_y L, \tag{8}$$

where H* is the distance from the inflection point of lower column to that of upper column.

For beam, adjacent to shear walls, yielding type;

$$V_y = \frac{1}{H^*}(1+\frac{3L_W}{4L_F})M_y, \; V_s = \frac{1}{H^*}(1+\frac{3L_W}{4L_F})M_s, \tag{9-a}$$

$$R_y = \frac{2}{9}\cdot\Phi_y L_F/(1+\frac{3L_W}{4L_F}), \tag{9-b}$$

where $3L_W/4L_F$ is derived from the assumption that the zero deflection point of beams adjacent to shear walls nearly coincides with the inflection point indicated by the point A such as shown in Fig. 3-c.

The effects of slabs on the flexural characteristics of beams are in this paper neglected.

Equivalent viscous damping coefficient--From the V-R hysteresis loop of Fig. 2, the equivalent viscous damping coefficient is derived by

$$h_{eq} = \frac{5+9s}{8\pi}\left(1-\frac{1}{\mu_{cr}}\right), \tag{10}$$

where

$$s = V_s/V_y\ , \quad \mu_{cr} = R_a/R_y\ . \tag{11}$$

Other critical characteristics--The critical ductility factor μ_{cr} is expressed as the function of the number of cycles until fracture n_B such as presented already by Eqs. 19, 20 in Chapter 5. The critical span ratio $\overline{H}_{cr}$ which was expressed by Eq. 6 in Chapter 5 also should be examined in order to avoid shear fracture of flexural members.

6-4. Application to Real Multi-Story Reinforced Concrete Structures With Shear Walls

The key plan of a structure which is selected as a sample is shown in Fig. 4, which is composed of rectangular rigid frames with 3.6m (11.8ft) story height and 7.0m (23.0ft) span length and of cantilever type shear walls. The aseismic capacity of such a structure subjected to earthquake loading in x-direction is investigated for the 9_{th} story counted from its top. The rigid frames in that story consist of the beams and columns such as shown in Fig. 5, whose sections are assumed to be all the same in that story.

The various characteristic values of beams and columns and their calculation processes are shown in Table 1, where the basic constants are assumed as follows:

$f'_c=200\text{kg/cm}^2(2840\text{psi})$, $f_y=4000\text{kg/cm}^2(57.0\text{ksi})$, ${}_c\varepsilon_u=4\text{x}10^{-3}$,

${}_s\varepsilon_y=2\text{x}10^{-3}$, $n_B=200\text{cycles}$, $W=1\text{t/m}^2\text{x}35\text{mx}21\text{mx}9\text{str.}=750\text{tx}9\text{str.}$,

$N=750\text{t}/24$pieces of column , $d'/h=d_s/h=0.1$.

The right hand side column in Table 1 shows the equations, by means of which calculations are performed. These values are so adopted as to fit the most popular and standard ones in Japan.

The hysteresis damping capacity of beams and columns and of beams adjacent to shear walls is computed as shown in Table 2, where 5.0 is assumed as the value of μ_{cr} of all the members from the technological point of view.

Finally, the aseismic capacity of the 9_{th} story of the structure is calculated and its aseismic safety is estimated on the assumption that $\alpha_o/g=0.3$ as shown in Table 3, where the two cases of yielding types, i.e., (a) columns in frames and beams adjacent to shear walls - yielding type and (b) beams in frames and beams adjacent to shear walls - yielding type. Furthermore, as complementary cases, (c) columns - and (d) beams - yielding types of the structure without shear walls are also considered.

Judging from Table 3, multi-story reinforced concrete structures seem to have the critical number of stories, from 8 to 9, independently of the existence of shear walls. Although the assumptions that the steady-state resonance occurs and that as the value of seismic weight the total weight of all stories is applied may be considered to cause the overestimation of earthquake loading, such a conclusion as achieved above is a result produced by comparing the maximum level of input with the maximum level of response in a state as critical as possible.

6-5. Concluding Remarks

The aseismic capacity of multi-story reinforced concrete buildings with and without cantilever-type shear walls is evaluated quantitatively in accordance with the aseismic concepts described in Chapter 3, and the followings are concluded:

(1) If the cantilever type shear walls show neither shear fracture nor flexural yielding, the yielding phenomenon of structure is caused by that of beams and frames in rigid frames and beams adjacent to shear walls.

(2) The aseismic capacity of such structures is able to be given by the left side of Eq. 6, i.e., the hysteretic damping capacity of flexural members.

(3) By the application of Eq. 6 to a real structure (see Figs. 4, 5), it is found that such a type of building has the critical number of stories, about 8 - 9, independently of the existence of shear walls.

The evaluation method of aseismic capacity and the estimation method of aseismic safety presented in the preceding chapters and this chapter are based upon the critical states and upon the maximum characteristic values of reinforced concrete structures and structural members. In the aseismic design of real structures, of course, there are yet more factors which should be taken into account. However, these concepts and methods described here should present the basis on the aseismic design and planning of reinforced concrete structures.

REFERENCES

(1) Cardan, B., "Concrete Shear Walls with Rigid Frames in Multistory Building Subjected to Lateral Loads," Jour. ACI, Vol. 58, No.3, Sept. 1961, pp. 299-316.

NOTATION

A = assumed inflection point of beams adjacent to shear walls

A_{Bi} = area of V-R hysteresis loop of beams adjacent to shear walls

A_{Fi} = area of V-R hysteresis loop of rigid frames

A_w = cross sectional area of shear walls

b = width of cross section of columns

d = distance from extreme compression fiber to centroid of tension reinforcement

d' = distance from extreme compression fiber to centroid of compression reinforcement

d_s = distance from centroid of tension reinforcement to the tensile face of columns

$F()$ = fracture criterion equation of concrete under combined normal and shear stresses

f'_c = compressive strength of concrete

f_y = yield strength of reinforcement

g = acceleration of gravity

H = story height of long columns and shear walls

H^* = distance between the inflection points of lower and upper columns

H_{cr} = critical story height

$\overline{H}$ = H/h

$\overline{H}_{cr}$ = H_{cr}/h

h = total depth of cross section of columns

h_{eq} = equivalent damping coefficient

i = number of stories

j = number of stories

K = stiffeness of story of structures

$[K_F]$ = matrix of K of rigid frames

$[K_B]$ = matric of K of beams adjacent to shear walls

$[K_W]$ = matrix of K of shear walls

L_F = span length of beams adjacent to shear walls

L_W = span length of cantilever type shear walls

M_s = slipping bending moment

M_y = yield bending moment

$[M]$ = matrix of lumped mass of structures

m = mass

N = axial force

N_o = ultimate strength of centrally loaded columns

n_c = number of cycles

R = relative story displacement angle

R_a = amplitude of R

R_y = yielding R.

$\{R\}$ = vector matrix of R

s = slipping coefficient ($=M_s/M_y$, $=M_{cs}/M_{cy}$)

t = thickness of shear walls, time

V = relative story shear force

V_y = yielding V

W = seismic weight of structures

W_j = W of j_{th} story

x = co-ordinate, suffix of direction

x_{nl} = ratio of distance from extreme compressive fiber to neutral axis to the total depth of cross section of columns

y = co-ordinate, suffix of direction

$\ddot{z}_G$ = acceleration of ground motion

α_o = acceleration amplitude of sinusoidal ground motion

γ = ratio of distance center-to-center of reinforcement on opposite faces of columns to h

δ = relative story displacement

${}_s\varepsilon_y$ = yield strain of reinforcement

${}_c\varepsilon_y$ = (idealized) yield compressive strain of concrete

${}_c\varepsilon_u$ = ultimate compressive strain of concrete

Φ_a = curvature amplitude

Φ_{acr} = critical Φ_a

Φ_y = yielding curvature

μ = ductility factor

μ_{cr} = critical ductility factor

χ = $({}_s\varepsilon_y / {}_c\varepsilon_y)$

ω = tensile reinforcing index

ω' = compressive reinforcing index

ω_0 = circular frequency of earthquake waves

Supplement

$\{\ddot{Y}\}$ = vector matrix of displacement acceleration relative to the ground

Table 1. Characteristic Values of Beams and Columns

		Column	Beam	Eq.
bxh	cmxcm	90x90	50x110	—
$f'_c bh$	ton	1940	—	—
$f'_c bh^2$	tm	1750	1452	—
N	ton	315	0	—
ω		0.10	0.14	—
$N_o/f'_c bh$		1.20	0.00	—
$h\Phi_{ac}$	$x10^{-3}$	31.40	—	19(Chap.5)
x_{n1}		o.162	—	3(Chap.5)
$M_y/f'_c bh^2$		0.1470	0.1120	2-a(chap.5)
$M_s/f'_c bh^2$		0.0152	0.1120	2-b(Chap.5)
s		0.1028	1.00	38
$h\Phi_y$	$x10^{-3}$	5.0	5.0	2-c(Chap.5)
$X(=N/N_o)$		0.135	0.00	7(Chap.5)
F(X)		0.1426	0.10	7(Chap.5)
$\bar{H}_{cr}$		2.60	2.84	6(Chap.5)
H_{cr}	cm	234	312	—
real H	cm	250	610	—
Frac. Mode		F	F	6(Chap.5)

Table 2. Resonance Capacity

	Column	Beam	Beam adjacent to Shear Walls	Eq.
R_y x10^{-3}	2.315	4.621	3.126	7, 8, 9
μ_{cr}	6.70	—	—	19,20 (Chap.5)
μ^*_{cr}	5.0	5.0	5.0	*assumed
h_{eq}	o.1866	0.446	0.446	—
V_y ton	207.0	103.6	89.0	7, 8, 9
$V_y h_{eq}$ ton	39.0	46.2	39.7	—

Table 3. Evaluation of Aseismic Capacity and Safety

Type of Str.	With Shear Walls				Without Shear Walls	
Yield.Typ.	(a)		(b)		(c)	(d)
Flexural Elements	Column	Beam adj. to Shear Walls	Beam	Beam adj.to Shear Walls	Column	Beam
$V_y h_{eq}$ ton	39.0	39.7	46.2	39.7	39.0	46.2
Number of pieces	20	4	(14+4x0.5) 16	4	24	20
$2\Sigma V_y h_{eq}$ ton	1918		1834		1872	1848
$(\Sigma W)\alpha_o/g$	2025		2025		2025	2025

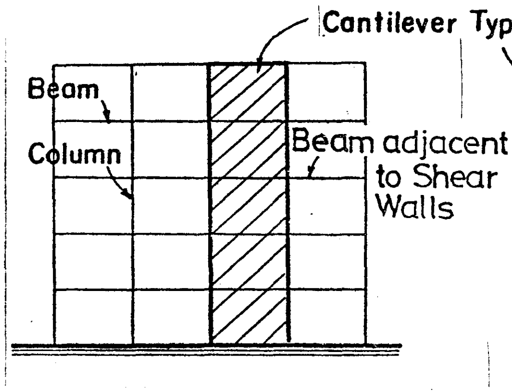


Fig. 1(a) Multistory Structures with Shear Walls

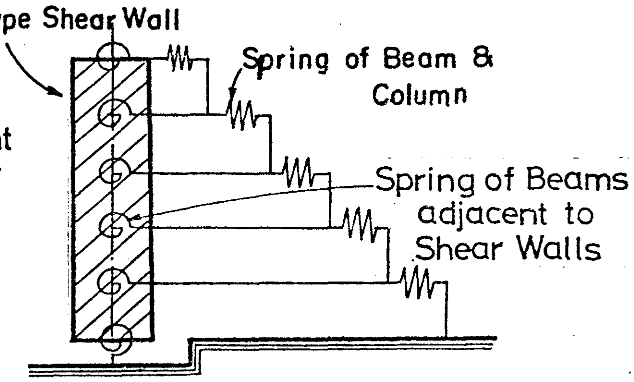


Fig. 1(b) Idealized Multistory Structure With Shear Walls [1]

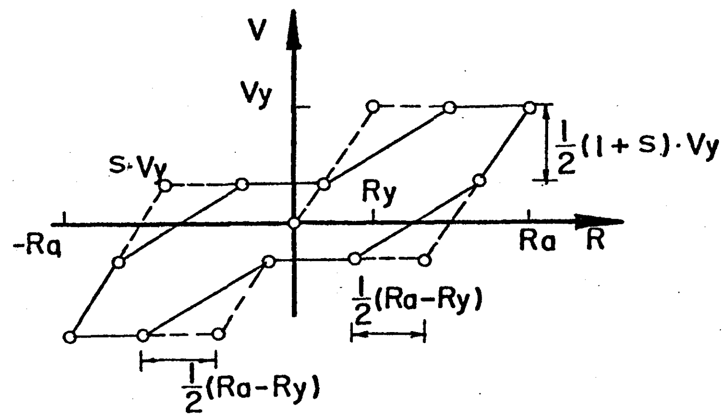


Fig. 2 Story Shear Force - Displacement Angle Hysteresis Loop of Flexural Members

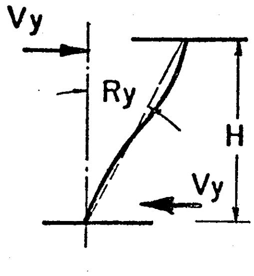


Fig. 3(a) Column Yielding Type

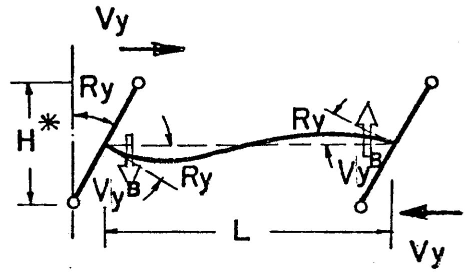


Fig. 3(b) Beam Yielding Type

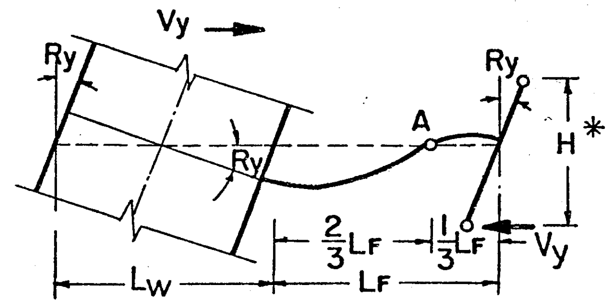


Fig. 3(c) Beam (Adjacent to Shear Walls) Yielding Type

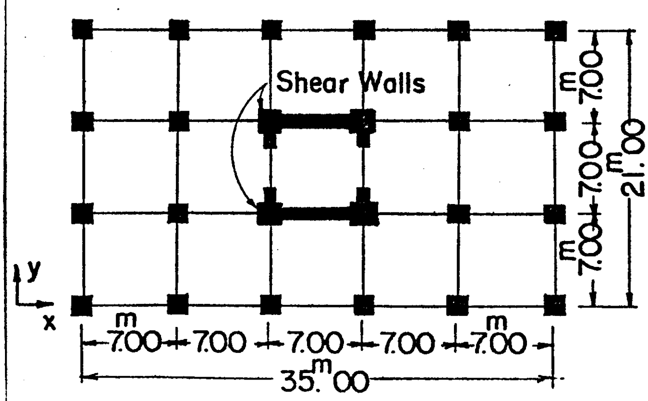
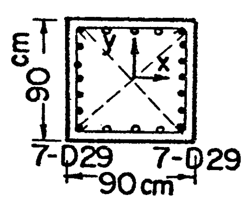
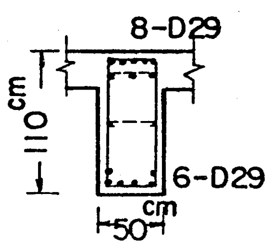


Fig. 4 Assumed Multistory Structure With Shear Walls



(a) Column



(b) Beam

Fig. 5 Assumed Cross Section of Columns and Beams

CHAPTER 7

SUMMARY AND CONCLUSIONS

7-1. Summary

It is proposed that the aseismic capacity of a reinforced concrete structure is the steady-state resonance capacity for an idealized one mass model of that structure subjected to a sinusoidal forced vibration (in Chapter 3). That resonance capacity is determined using the known characteristics of the hysteresis loops and the stability in strength with cycling of typical reinforced concrete elements. For this evaluation reinforced concrete members are classified, according to their modes of deformation and failure under monotonic loading, into three types of aseismic elements: Long columns, short columns, and shear walls (in Chapter 2). Structures are divided into two types: rigid and flexible. A diagram is developed showing the interaction surface in three dimensional space between shear force amplitude, equivalent viscous damping and the natural period of vibration. The range of aseismic characteristics for the different elements and structures are shown on that diagram. Response acceleration spectra for typical earthquake inputs are collated with the response indicated by that diagram. That interaction surface is shown to represent a reasonable basis for estimating the aseismic safety of reinforced concrete structures.

Based upon the fundamental concept of aseismic capacity of classified reinforced concrete structures, the aseismic capacity and safety are investigated regarding two types of reinforced concrete structures, i.e., rigid structures with shear walls and flexible structures without shear walls (in Chapter 4). First, the aseismic characteristics of medium or low rise reinforced concrete structures, i.e., wall ratios, natural periods and critical number of stories are indicated as contourlines in $\alpha/\gamma - \beta/\gamma$ plane, where α, β and γ show the number of pieces of long columns, short columns and shear walls composing the structures, respectively. As for rigid structures which should resist against earthquakes through story shear resisting capacity, it is shown that wall ratio is the most significant factor for the aseismic capacity and safety. As for flexible structures which should resist against earthquakes through the hysteretic damping capacity, analytical expressions of the hysteretic damping capacity and the fatigue characteristics of flexural members composing the structures are derived and applied to a 4-story reinforced concrete structure with column-yielding type in order to estimate its aseismic capacity and safety.

In order to evaluate the aseismic capacity and safety, the fundamental concepts and methods introduced in the preceding chapter are applied to two types of reinforced concrete structures, i.e., low rise buildings with asymmetric shear walls (in Chapter 5) and multi-story buildings with shear walls (in Chapter 6). As for the structures with asymmetric shear walls, their aseismic capacity is estimated by means of hysteretic damping capacity at steady-state resonance of torsional vibration. As for the multi-story structures with shear walls, their aseismic capacity is evaluated by means of the hysteretic damping capacity of flexural members except shear walls, i.e., beams and columns. By applying these principles to the real types of reinforced concrete structures, the maximum and critical values of aseismic capacity are simply given, and it is shown that these values are able to become the basis for the aseismic design.

7-2. Conclusions

From the considerations in this paper, the following concluding remarks were obtained:

(1) A reinforced concrete structure composed of three kinds of aseismic elements, i.e. long columns, short columns and shear walls is able to be classified into three kinds of fracture modes, i.e. LC-, SC- and SW-Fracture Modes, and furthermore into two kinds of aseismic types, i.e. rigid and flexible ones (Figs. 15, 16 in Chapter 2).

(2) When steady-state resonance is adopted as an ultimate vibration state of structures subjected to strong earthquakes, Resonance Capacity $2V_a h_{eq}$ is able to be introduced as an criterion for the evaluation of the aseismic capacity of reinforced concrete structures and structural members (Eq. 12 in Chapter 3).

(3) Based upon the Resonance Capacity criterion, the ultimate aseismic capacity of reinforced concrete structures is made visible in V_a-T_e-h_{eq} space (Fig. 10 in Chapter 3). Fig. 10 in Chapter 3 shows that rigid

and flexible type structures resist against earthquake inputs through story shear capacity and hysteretic energy absorption, respectively.

(4) Using three kinds of aseismic elements with standard and general sizes and dimensions, the fracture modes, wall ratios, critical number of stories and natural periods of reinforced concrete structures are shown and arranged in a common plane (Fig. 8 in Chapter 4).

(5) Rigid type structures are considered to be subjected to the dead loads as story shear force at destructive earthquakes. As for flexible type structures, i.e. LC-fracture mode, the critical story shear coefficient can be reduced to about 0.5-0.6 due to the hysteretic energy absorption capacity and low cycle fatigue characteristics (Chapter 4).

(6) Low rise reinforced concrete structures with asymmetrically arranged shear walls are able to be regarded as laterally torsional oscillators (Figs. 1, 7 in Chapter 5), and the evaluation method of aseismic capacity is essentially analogous to the horizontal sway type structures without shear walls.

(7) Multi-story reinforced concrete buildings are able to be considered as flexible type structures, and the critical number of stories is about 8~9, when the shear and flexural failures of cantilever type shear walls in the structures are avoided (Fig. 1 in Chapter 6).

In this study, reinforced concrete structures and structural members with standard and general types and sizes were adopted as the objects of investigations. From now on ideal reinforced concrete structures with optimum styles, sizes and dimensions should be researched on the basis of the criteria proposed in this paper for the evaluation of the ultimate aseismic capacity.

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