Earthquake Engineering, Tenth World Conference © 1992 Balkema, Rotterdam. ISBN 90 5410 060 5

Experimental and analytical studies on the seismic behavior of reinforced concrete frame-wall structural building

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ABSTRACT: The experimental and analytical studies were carried out to estimate the seismic safety margin of a 25 storied reinforced concrete residential building designed as a frame-wall structure. Cyclic loading tests of column members, sub-assemblages and shear walls were conducted. Using the restoring force characteristics of the members determined under these experimental results, the nonlinear static and dynamic analyses were carried out. The response of this building structure to severe earthquake ground motions was less than seismic design criteria and was much less than its seismic capacity obtained by the experimental studies.

1 INTRODUCTION

The frame-wall structural system has advantages in the structural design resulting from high stiffness and strength of shear walls. Design guidelines for a high-rise frame-wall structural system, however, have not been established due to the lack of appropriate experimental data. Therefore, in order to estimate the seismic safety margin of a 25 storied reinforced concrete residential building designed as a frame-wall structure, the analyses and experiments were carried out.

The designed structure is 71.35 meters high and its standard plane form is hexagon with the hexagonal hole at the center of the plane. Interior and exterior frames are combined by shear walls arranged radially as shown in Fig.1. The cross section of the columns is circular form, and the longitudinal reinforcements to resist the large tensile axial stress due to lateral forces are arranged at the center of the cross section in the lower floors. The maximum compressive strength of concrete employed in the structural design is $360kgf/cm^2$. The yield strength of longitudinal reinforcement is $4000kgf/cm^2$ and the yield strength of spiral hoop bar used in columns is $13000kgf/cm^2$. The seismic design criteria for LEVEL1 and LEVEL2 earthquakes were employed to estimate the seismic safety (Table 1). The maximum velocities for LEVEL1 and LEVEL2 earthquake ground motions were normalized to 25 and 50 cm/sec, respectively.

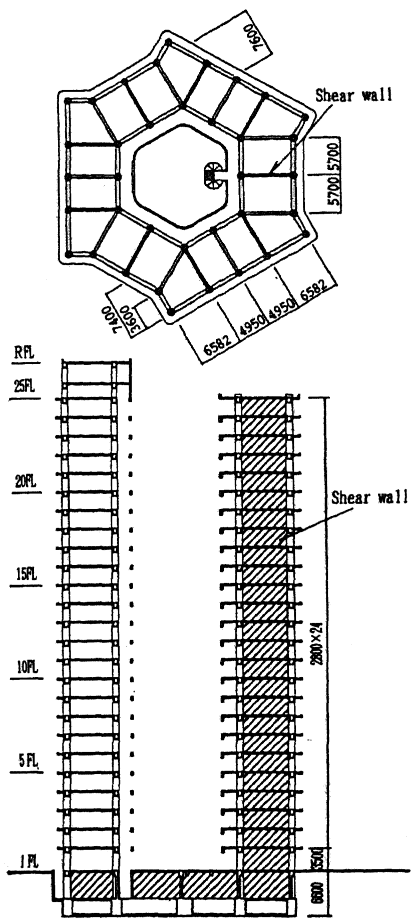


Fig.1 Frame-wall structure

2 EXPERIMENTAL STUDIES

The preliminary analyses were carried out, in which the frame model was used for nonlinear static analyses subjected to lateral forces and the lamped-mass spring model and frame model were used for nonlinear dynamic response analyses to LEVEL1 and LEVEL2 earthquake ground motions. Cyclic loading tests of column members, sub-assemblages and shear walls were conducted. The specimens employed in this experimental study were chosen to focus on the parts of the building structure where the comparatively large story drifts were obtained by the preliminary nonlinear static and dynamic analyses.

2.1 Cyclic Loading Tests of Column Members

In order to evaluate the ultimate shear strength of circular column members using ultra high strength bars as the spiral hoop, eight specimens were provided for the experiments. The arrangements and dimensions of the models of column members are as shown in Fig.2. The parameters of experiments were the axial stress and the shear reinforcement ratio. The shear span ratio was equal to 1.0. The compressive strength of concrete and the yield strength of reinforcing bars are as shown in Table 2.

The failure mode for CC08 applied tensile axial stress was the flexural failure, for CC04 and CC07 applied higher level of compressive axial stress was the shear compression failure at the center of columns and for the other specimens was the shear compression failure at the ends of columns. Comparing the ultimate shear strength of the tests with one calculated by Eq.(1) shown in Fig.3, the calculated values of two specimens applied higher level of compressive axial stress overestimated the ultimate shear strength of the specimens.

$$Q_u = \left\{\frac{0.0679 P_t^{0.23}(F_c + 180)}{M/(Qd) + 0.12} + 2.7P_w\sigma_{wy} + 0.1\sigma_0\right\}\frac{7}{8}b_e d \quad , \tag{1}$$

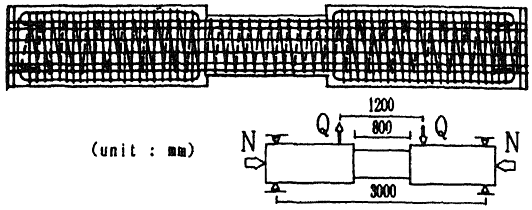
where Q_u = ultimate shear strength
P_t = tension reinforcement ratio
F_c = compressive strength of concrete
$M/(Qd)$ = shear span ratio
P_w = shear reinforcement ratio
σ_{wy} = yield strength of the shear reinforcement
σ_0 = axial stress
b_e ,d = width and effective depth of section, respectively

The stress of shear reinforcing bar at the ultimate shear strength was about 6000kgf/cm². The value of 6000kgf/cm² is employed in Eq.(1), instead of yield strength of the shear reinforcing bar, the ultimate shear strength could be estimated by Eq.(1) as shown in Fig.3.

2.2 Cyclic Loading Tests of Sub-assemblages

Table 1 Criteria for LEVEL1 and LEVEL2 earthquake

	LEVEL1	LEVEL2
Maximum story drift	1/200 rad.	1/100 rad.
Maximum ductility factor for story	1.0	2.0
Maximum ductility factor for element	1.0	tention column 4.0 beam 4.0 shear wall 2.0



Specimen	Axial stress	Cross section
CC01 CC02 CC03 CC04	0 0.15Fc 0.30Fc 0.45Fc	400 12-D16 P_g =1.90% ○ 6.4φ@65 P_w =0.260%
CC05 CC06 CC07 CC08	0 0.30Fc 0.60Fc −0.15Fc	400 16-D16 P_g =2.53% ○ 6.4φ@55 □ 6.4φ@55 P_w =0.615%

Fc : Compressive strength of concrete

Fig.2 Arrangements and dimensions of column members

Table 2 Compressive strength of concrete and yield strength of reinforcing bars

Specimen	$_c\sigma_B$ (kgf/cm²)
CC01	317
CC02	447
CC03	466
CC04	388
CC05	313
CC06	458
CC07	461
CC08	494

Diameter of bar	$_s\sigma_y$ (kgf/cm²)
D16	7490
7.4 φ	13000

$_c\sigma_B$: Compressive strength of concrete
$_s\sigma_y$: Tensile yield strength of reinforcing bars

Four half-scale sub-assemblages represented an interior portion of the 1st and 9th story were provided for the experiments. The arrangements and dimensions of the models of sub-assemblages are as shown in Fig.4. Compressive axial stress were maintained as the constant value during a test. The value of compressive axial stress of the column was 50kgf/cm² in CP01 and CP02, 60kgf/cm² in CP03 and CP04. CP02 and CP04 have the transverse beam and slab. The compressive strength of concrete and the yield strength of reinforcing bars are as shown in Table 3.

Typical hysteresis curves obtained by the experiments are as shown in Fig.5 and Fig.6, respectively. Plastic hinges were formed at the end of beams in CP01 and CP02, and at the bottom of column and the end of beams in CP03 and CP04. Each sub-assemblages behaved in a ductile manner. The ductility factor at the ultimate strength was about 4 in CP02 and 6 or more in CP04. As shown in Fig.5 and Fig.6, the analytical skeleton curves were well following the test results of all specimens, though the ultimate strength was slightly underestimated.

In order to estimate the capacity of development with 90-deg hook in the beam column joint under the tensile axial stress in the column, three half-scale sub-assemblages represented an exterior portion of the 3rd and 9th story were provided for the experiments. The arrangements and dimensions of the models of sub-assemblages are as shown in Fig.7. Development length of all specimens was 30d where d was the bar diameter. Tensile axial stress was applied to the columns of all specimens. The values of tensile stress for the longitudinal reinforcing bars of the column were 2000kgf/cm² in EP01 and EP02, 1000kgf/cm² in EP03. The compressive strength of concrete and the yield strength of reinforcing bars are as shown in Table 4.

Typical hysteresis curves obtained by the tests are as shown in Fig.8. Plastic hinges were formed at the end of beams in all specimens. The hysteresis curves for all specimens showed excellent stability even at large displacement levels(1/20rad.). The capacity of development with 90-deg hook was sufficient.

2.3 Cyclic Loading Tests of Shear Walls

Two shear walls with shear span ratio of 2 or 3 represented the 1st and 2nd story were provided for the experiments. Each shear walls were one-fourth scale and consisted of side columns and beams. The arrangements and dimensions of the models of shear walls are as shown in Fig.9. In order to adjust to the overturning moment at the first floor obtained by the preliminary analyses, vertical loads were applied at the ends of upper stab as shown in Fig.9. The compressive strength of concrete and the yield strength of

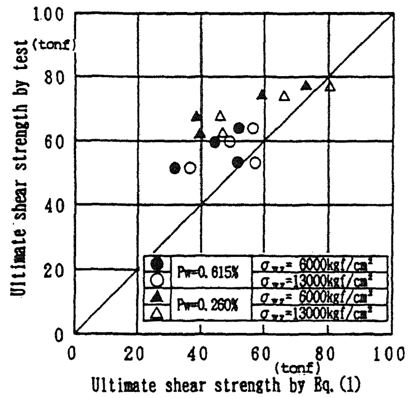


Fig.3 Ultimate shear strength

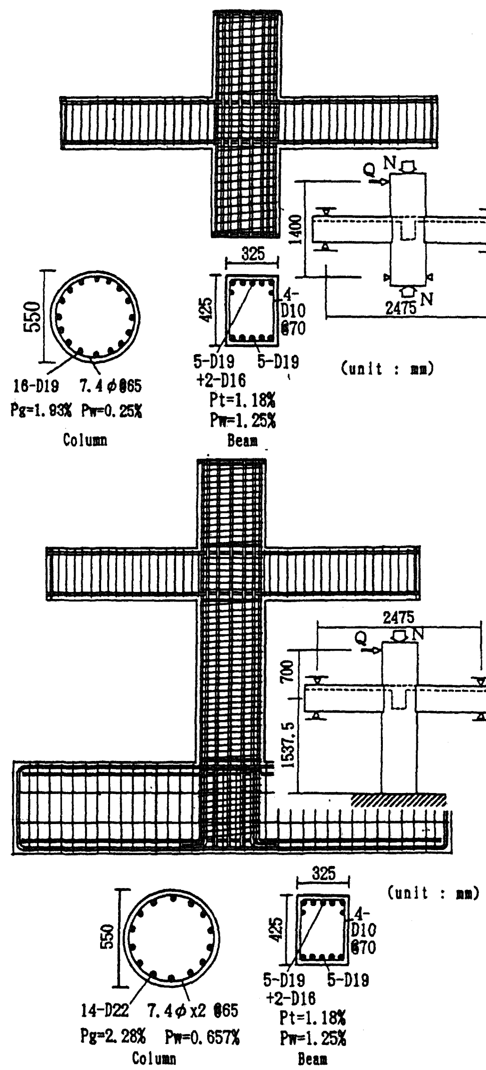


Fig.4 Arrangements and dimensions of sub-assemblages

Table 3 Compressive strength of concrete and yield strength of reinforcing bars

Specimen	$_c\sigma_B$ (kgf/cm²)
C P 0 1	3 0 8
C P 0 2	3 0 8
C P 0 3	3 7 2
C P 0 4	3 6 7

Diameter of bar	$_s\sigma_y$ (kgf/cm²)
D 2 2	4 5 7 0
D 1 9	4 5 5 0
D 1 6	4 7 0 0
D 1 0	3 9 6 0
D 6	3 8 5 0
7.4 ϕ	1 3 0 0 0

$_c\sigma_B$: Compressive strength of concrete
$_s\sigma_y$: Tensile yield strength of reinforcing bars

Table 4 Compressive strength of concrete and yield strength of reinforcing bars

Specimen	$_c\sigma_B$ (kgf/cm²)
E P 0 1	4 3 5
E P 0 2	4 6 5
E P 0 3	2 9 9

Diameter of bar	$_s\sigma_y$ (kgf/cm²)
D 2 2	4 1 0 0
D 1 9	4 5 0 0
D 1 6	4 7 0 0
D 1 0	3 9 6 0
7.4 ϕ	1 3 0 0 0

$_c\sigma_B$: Compressive strength of concrete
$_s\sigma_y$: Tensile yield strength of reinforcing bars

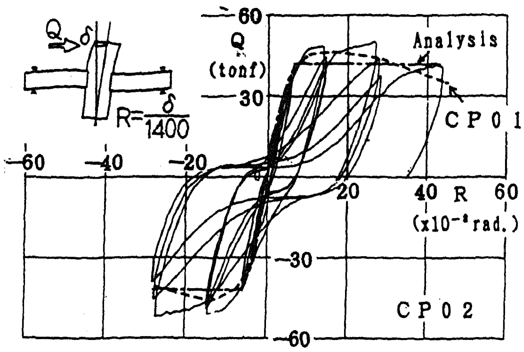


Fig.5 Hysteresis curves

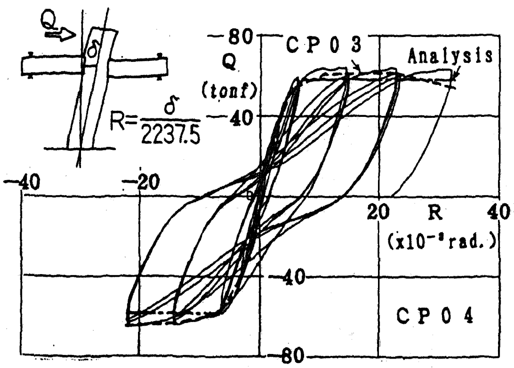


Fig.6 Hysteresis curves

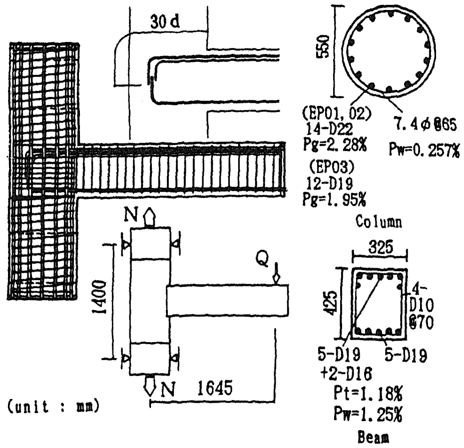


Fig.7 Arrangements and dimensions of sub-assemblages

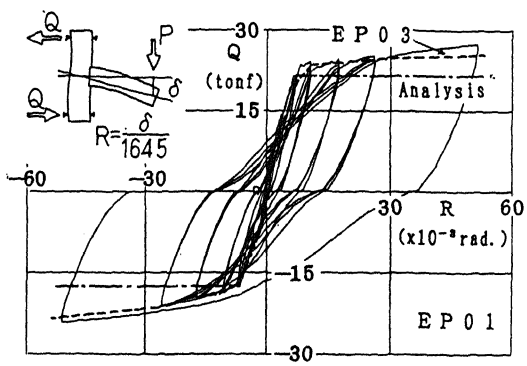


Fig.8 Hysteresis curves

reinforcing bars are as shown in Table 5.

Typical crack pattern at 1/75rad. and hysteresis curves obtained by the tests and the analyses are as shown in Fig.10 and 11, respectively. The flexural cracks occurred all over the columns and the shear cracks occurred all over the shear wall. Finally, crushing of the concrete occurred at the bottom of columns. Plastic hinges were formed at the bottom of shear wall. The deflected angle at occurrence of the yielding in this region coincided with the analysis. Each shear walls behaved in a ductile manner to 1/75 rad. or more of story drift. The shear walls had sufficient seismic capacity to the maximum response displacement of LEVEL2 earthquake and the displacement at the ultimate strength of the structure. The analytical hysteresis curves were determined to be well following the experimental hysteresis curves as shown Fig.10.

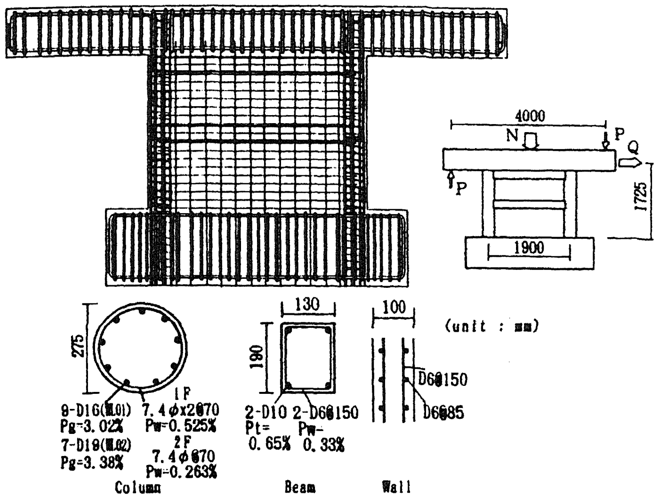


Fig.9 Arrangements and dimensions of shear walls

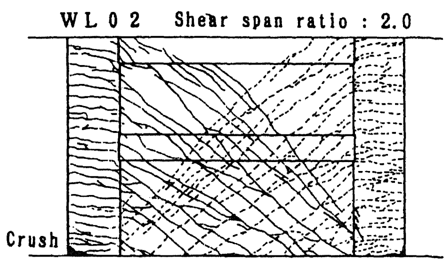


Fig.10 Typical crack pattern

Table 5 Compressive strength of concrete and yield strength of reinforcing bars

Specimen	$_c\sigma_B$ (kgf/cm²)
WL01	342
WL02	369

Diameter of bar	$_s\sigma_y$ (kgf/cm²)
D19	4330
D16	4600
D10	4060
D6	4110
7.4 φ	13000

$_c\sigma_B$: Compressive strength of concrete
$_s\sigma_y$: Tensile yield strength of reinforcing bars

3 ANALYTICAL STUDIES

The analyses were carried out by the same procedures as the preliminary analyses, in which the restoring force characteristics of the element models were determined under these experimental results.

3.1 Nonlinear Static Analysis

Tri-linear type of skeleton curves determined under these experimental results were used for the nonlinear static analysis. The inverted triangular type was employed as the lateral force distribution.

The typical state of the yield hinges are as shown in Fig.12. Plastic hinges were formed at the end of beams, at the columns subjected to tensile axial force and at the bottom of shear walls. These yield hinges were assessed to maintain the sufficient ductility by the previous experimental studies.

3.2 Nonlinear Dynamic Response Analysis

Degrading tri-linear type of hysteresis models determined under these experimental results were used for the moment-rotation relation of columns, beams and shear walls, and origin-oriented tri-linear type for the shear force - shear deflection angle relation of shear walls. The damping coefficient proportion to the instantaneous stiffness of the structure, having the initial value of 3%, was assumed. The input ground motions were 1940 El Centro NS, 1952 TAFT EW, 1956 Tokyo 101 NS and 1968 Hachinohe NS, and they were normalized to have the maximum velocities of 25 and 50 cm/sec for LEVEL1 and LEVEL2 earthquake ground motions, respectively.

The distributions of the maximum story drifts obtained by the nonlinear dynamic response analyses are as shown in Fig.13. The obtained maximum story drifts were 1/300rad. in LEVEL1 and 1/120 in LEVEL2. They were much smaller than the capacity of frames obtained by these experiments for the structure. Plastic hinges were generated at the end of beams located in the middle stories.

4 CONCLUSIONS

From the following results obtained by the

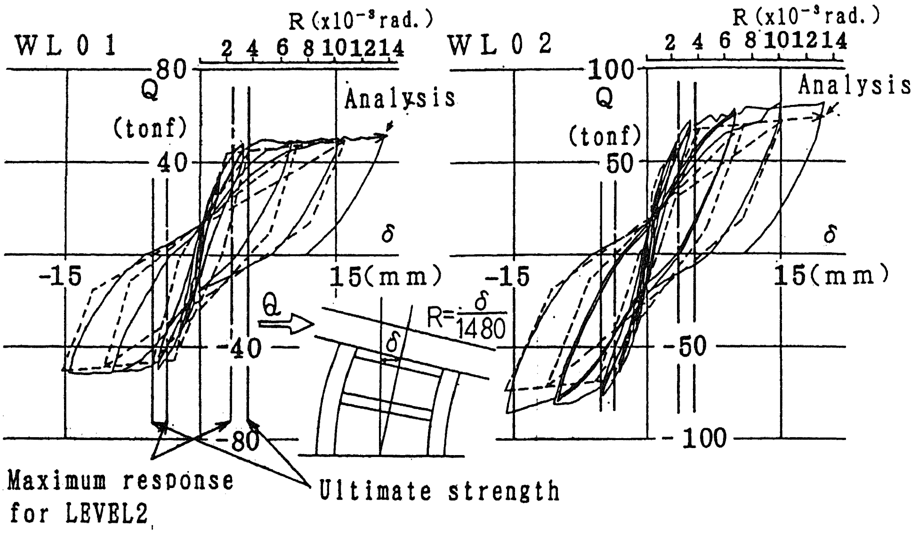


Fig.11 Hysteresis curves

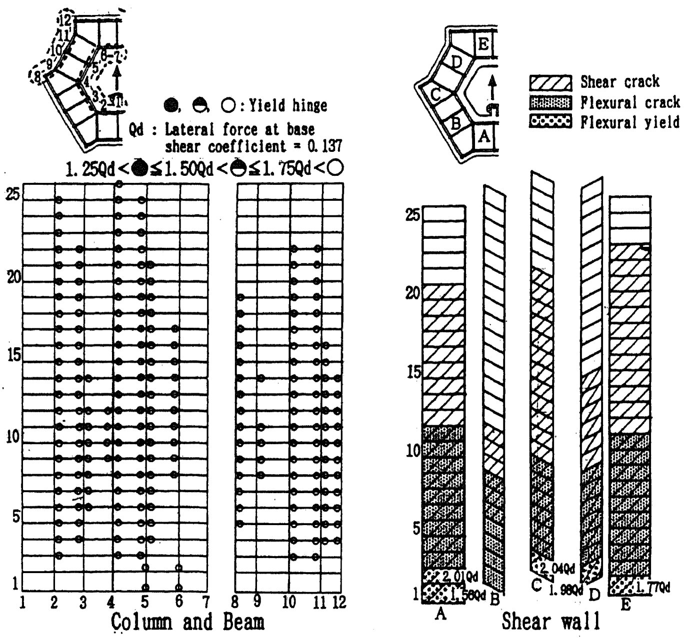


Fig.12 Typical yield hinges distribution

experimental and analytical studies, the 25 storied reinforced concrete residential building was designed as a frame-wall structure which had sufficient seismic capacity to severe earthquake ground motions.

1.The ultimate shear strength of the circular column members was evaluated by the present experimental formula under the upper limit of the yield stress of the ultra high strength spiral hoop bar.

2.The sub-assemblages and the shear walls behaved in a ductile manner and showed the flexural failure.

3.The failure mechanism of this building structure was formed by the flexural yield at the end of the beams and the tensile columns and at the bottom of the shear walls.

4.The maximum values of response analyses of this building structure to severe earthquake ground motions was less than seismic design criteria and was much less than seismic capacity obtained by the experimental studies.

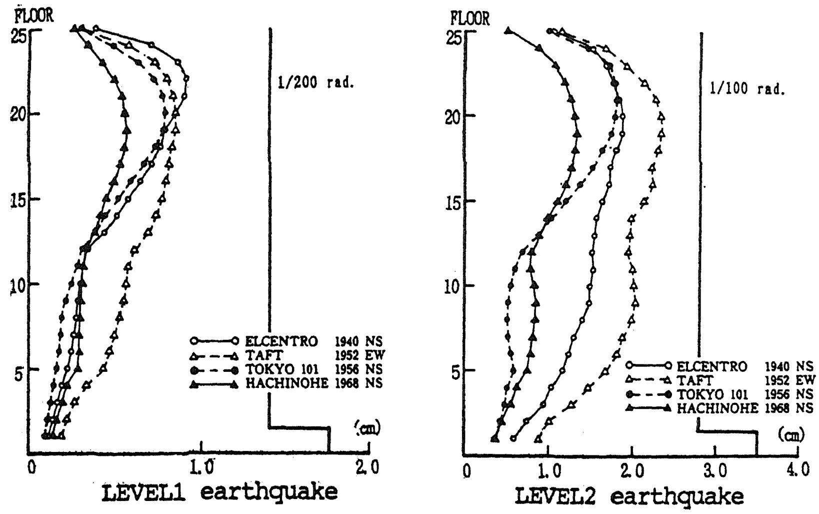


Fig.13 Maximun story drift by dynamic response analysis

5 ACKNOWLEDGMENTS

The authors wish to express their gratitude to Professor Ikuhiro Mtsuzaki who provided valuable advices and help in this work.