EXPERIMENTAL AND ANALYTICAL STUDIES ON THE SEISMIC BEHAVIOR OF R/C MOMENT RESISTING FRAMES WHICH HAVE WIDE COLUMNS

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SUMMARY

The seismic resistance of reinforced concrete wall-type moment resisting frames, which have wide columns like wall panel structures, was experimentally and analytically investigated to obtain design guidelines. Cyclic loading tests were conducted for models of subassemblages and members in medium to high-rise frames, and their ductile behavior was verified. It was indicated by the nonlinear static and dynamic analyses that the response of the structural system to severe ground motions would be much less than its seismic capacity observed in the experimental studies.

INTRODUCTION

Because of advantages in the architectural design and construction, wall-type moment resisting frames of reinforced concrete, which have wide columns like walls, are proposed for medium to high-rise residential buildings (Fig.1). It is essential in these frames that a beam has identical thickness to that of adjacent column so that larger interior spaces of the building may be provided and formworks in the construction for beam-column joints may be simplified. The structural system also has advantages in the structural design resulting from high stiffness and strength of wide columns. A weak-beam type failure mechanism is easily realized and the high stiffness of a total structure reduces the displacement during a severe earthquake.

Design guidelines for such a structural system, however, have not been established due to the lack of appropriate data. Therefore, a series of experimental and analytical studies were undertaken to examine the hysteretic behavior of the frames and the dynamic response of a total structure to severe earthquakes. Cyclic, loading tests were conducted for frame subassemblages, columns and T-beams which represented critical regions in a prototype building subjected to lateral forces (Fig.2). The obtained results were investigated with respect to their failure pattern, ultimate strength, energy dissipation and ductility. In the analytical program, the response of model frames to static lateral forces or to some specific ground motions were investigated. The nonlinear static analysis used a beam model with inelastic rotational springs at its ends. The nonlinear dynamic analysis was conducted using inelastic models for the story level or the member level.

EXPERIMENTAL STUDIES OF FRAME SUBASSEMBLAGES AND MEMBERS

Cyclic Loadin Test of Frame Subassemblages

Eight simple frame subassemblages were provided for the experiments (Figs. 2 to 4). The structure represented an interior portion of the first story of

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10 to 15-stories prototype buildings. Each frame was one-fourth scale and consisted of a column and beams of adjacent story. The beam represented a half length of an actual beam and its far end was suported by a pin and a roller (Fig.4). To reproduce a single curvature moment distribution in an actual column due to high overturning moment, lateral forces were applied to the upper column beyond the story under consideration.

The factors for the test program, mainly those related to columns, were 1) aspect ratio (D/B; where D and B were the depth and width, respectively), 2) shear span ratio (M/QD), 3) axial stress (N/BD) and 4) arrangement of reinforcing bars. The value of these factors was D/B = 2, 3, 5, M/QD = 1.5, 2.0, N/BD = 0.20, 0.29fc' where fc' was compressive strength of concrete (31 to 33 MPa). When these factors were determined, three levels of ultimate strength of the frame, i.e., 0.07, 0.11 and 0.15fc' in terms of the nominal shear stress in a column, was taken into account. Typical arrangement of reinforcing bars is shown in Fig.3. Web reinforcement was provided for shear forces developed by the ultimate moments at critical sections. In Frames 7 and 8, diagonal web reinforcement was provided together with longitudinal reinforcement. The yield strengths of reinforcing bars of D6(#2), D10(#3), D13(#4) and D16(#5) were 390, 400, 430 and 380 MPa, respectively.

Typical failure mode and hyteresis curves observed in the tests are shown in Figs.5 and 6. Plastic hinges were formed at the bottom of column and at the ends of beams. The yielding in these regions occurred at similar level of story displacement. Each frame behaved in a ductile manner sustaining large post-yielding displacement. In all the cases, finally the column failed in flexural compression and buckling of longitudinal reinforcement (Fig.6). In some cases beams were followed by the brittle failure, though at the range of large displacement. The envelope of hysteresis curves is summarized in Fig.7. The ultimate displacement of each frame, defined as the displacement at the maximum load or larger displacement at descending load of 80% the maximum, is marked in the figure. Frames 1 and 2 with thicker column dimension indicated the largest displacement ability. Other frames sustained the ultimate displacement from 0.018 to 0.040 radian and the ductility factor from 5 to 8, while the obtained nominal ultimate shear stress was 0.093 to 0.147fc'. Frame 7 failed in buckling of longitudinal reinforcement at relatively small displacement because of the small amount of these bars.

<u>Cyclic Loading Test of T-Shaped Beams and Exterior Columns</u>

For the former test program, seven half-scale T-beams were constructed. In the first two beams, the outer longitudinal reinforcement was arranged so as to run outside or alternatively inside the column reinforcement in the joint (B01 and B02). As the beam also had narrow width, it was followed by rather small area of confined concrete and thick concrete cover in the latter case of configuration, while larger confined area was obtained in the column section. It was considered be necessary in this case to confine the thick concrete cover to improve the ductility of the beam. Thus, the other test beams were provided with supplemental stirrups, in addition to the main stirrups, at their critical sections. The amount of these stirrups was the main factor for the program (Table 2). Ultra-high strength bars having the yield strength about 1500 MPa were used for these stirrups except the third beam with normal bars. The yield strengths of other bars was 310 to 400 MPa, and the strength of concrete was 21 to 23 MPa. The beams were tested under a simple beam type loading method.

Each beam indicated a ductile behavior having rich hysteresis curves (Fig. 9). The failure occurred at its bottom due to crushing of concrete and buckling of main bars when the slab was in tension. The observed ductility, though it was much larger when the slab was in compression, was affected by the amount of supplemental stirrups (Fig.10). In case the largest amount (B03 and B06), the largest ductility, about 6 as an average, was observed while the factor was less than 5 without these stirrups. Here the factor was defined as the ratio of ultimate displacement, determined by the previously described rule, to the yield displacement. The effects of strength of the stirrups and configuration of main bars appeared to be minor.

The design calculation of a 15-stories prototype frames indicated that exterior columns would be subjected to alternate axial tension and high compression induced by lateral forces as well as gravity loads. To investigate the behavior of these columns, three one-third scale identical columns were tested under different condition of axial and lateral forces (Fig.11). The loading condition is shown in Fig.11 in terms of shear span ratio and axial stress. Where the last combination of the factors (alternately, $M/QD = 2.0$ and 4.7, $N/BD = -0.22fc'$ and $0.53fc'$ where $fc' = 22.5$ MPa) corresponds to that at the hinge mechanism of the prototype structure. The obtained hysteresis curves of C3 column and their envelope are shown in Figs.12 and 13. C1 column, subjected to axial tension, sustained large displacement, 0.07 rad., without loss of load capacity. Although C3 column, subjected to the severest loading condition, finally failed in crushing of concrete due to the high axial compression, its behavior was ductile until the displacement of 0.03 rad. As compared in Fig.13, its displacement ability was similar to or larger than those of interior columns in the previous tests of subassemblages.

ANALYTICAL STUDIES OF PROTOTYPE FRAMES

Nonlinear Analysis of Model Frames Subjected to Lateral Forces

The hysteresis curves of previous test frames were analyzed by the procedures to use the inelastic beam element shown in Fig.14 [2]. Trilinear type hysteresis models were used for the moment-rotation relation of the element. The failure mode and the values to control the model were determined based on the existing test data of components. As the plots in Fig.15 indicate, the analytical curves were well following the test results, though the strength was slightly underestimated. The displacement at occurrence of yielding or failure was also well estimated.

Using the same procedures, the relation of displacement of columns and rotation of beam ends were analyzed for the case with a 11-stories model frame subjected to inverted triangular type lateral forces (Fig.16). It was detected that the rotational ductility of beams at middle stories was larger by 20 to 50% than the displacement ductility of the bottom column. This suggested that beams at upper stories in this type of structure would sustain much rotation because of its short span and cantilever displacement of wide columns.

Nonlinear Dynamic Response Analysis of Model Frames

Two types of nonlinear dynamic analyses were made for 15-stories model frames having different strength of beams (Fig.17). In Frame No.1, the yield strength of beam was reduced along the height of frame by up to 30%, while it was more reduced to 42 % in another frame No.2. The analysis of the frames by the previous procedures (Fig.18) indicated that their ultimate base shear coefficients were 0.41 and 0.38. Two hysteresis models were used for the response analysis (Fig.19). In the story level analysis, a tri-linear type force-displacement relation was assumed for each story (shear model). In a rigorous analysis, the tri-linear hysteresis model was used for each framing element (member model). The damping coefficient proportional to the instantaneous stiffness of the structure, having the initial value of 5%, was assumed. The selected ground motions were those recorded at the earthquakes of 1940 El Centro, 1968 Tokachi-oki and 1978 Miyagiken-oki, and they were factorized to have the maximum acceleration of 0.45g. As Fig.20 indicates, the shear model generally provided larger response than that by the member model because of the weak-beam type structural system, as suggested by Takizawa [4]. The obtained maximum displacement and ductility were 0.013 rad. and 1.5 for the shear model, while they were 0.010 rad. and 1.0 in another model. They were much smaller than the capacity of frames observed in the previous tests for the structure. The effect of the change of beam strength was minor.

CONCLUSIONS

As a result of the experimental and analytical investigations of medium to high-rise wall-type moment resisting frames, the following conclusions will be made. 1) The structural system will behave in a ductile manner until the displacement 0.02 rad. or more, and ductility more than 5, when ductile beams are provided and interior columns meet the following conditions, i.e., aspect ratio 5 or less, axial compression less than 0.30fc' and nominal shear stress less than 0.15fc'. 2) More ductility of beams will be obtained by the addition of supplemental stirrups to confine the concrete cover. 3) The behavior of exterior columns subjected to high axial compression will be as much ductile as those of interior columns. 4) Although the response of the structure to severe ground motions of 0.45g level may exceed the yield displacement, it will be much less than its capacity verified by the experiments.

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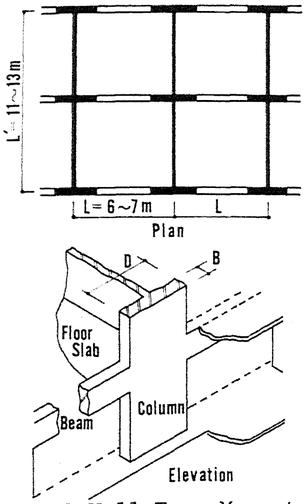
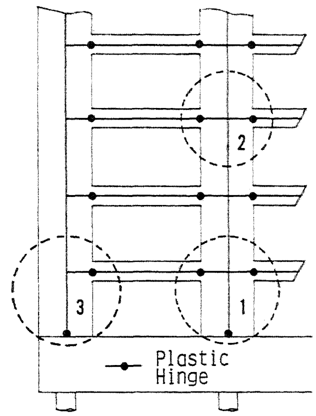


Fig.1 Wall-Type Moment Resisting Frames



1 Frame Subassmblage Tests
 - Aspect Ratio of Column
 - Axial Forces
 - Shear Span Ratio
 - Web Reinforcement

2 Beam Tests
 - Web Reinforcement
 - Supplemental Stirrups
 - High Strength Bars for Stirrups

3 Exterior Column Tests
 - Variable Axial Force
 - Tensile Axial Force
 - Shear Span Ratio

Fig.2 Test Program

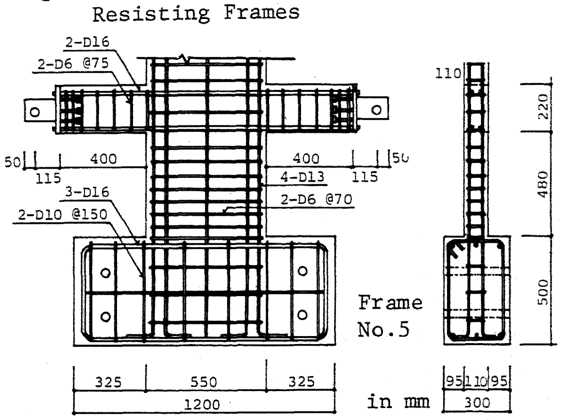


Fig.3 Reinforcement of Test Frame

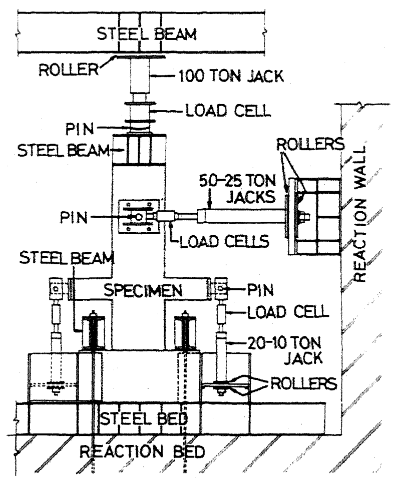


Fig.4 Test Setup

Table 1 Test Frames

No.	Column			Beam		
	Section and Aspect Ratio	Reinforcement Main	Reinforcement Web	Section	Reinforcement Main	Reinforcement Web
1	10 x 20cm², 2.0	0.71%	0.64%	14 x 22cm²	1.86%	0.61%
2	14 x 42cm², 3.0	0.73	0.61	"	1.46	0.46
3,4	11 x 55cm², 5.0	0.71	0.58	11 x 22cm²	1.90	0.78
5	" "	0.84	0.83	"	"	"
6	" "	0.59	0.83	"	"	"
7	" "	0.24*	0.58	"	1.85	0.97
8	" "	0.71*	0.39	"	"	"

• Column Shear Span Ratio : 2.0(No.1 - No.5), 1.5(No.6 - No.8)
• Column Axial Stress : fc'/5.0 (No.5), fc'/3.5 (the other)
* X-shaped reinforcement (2-D16 (#5))

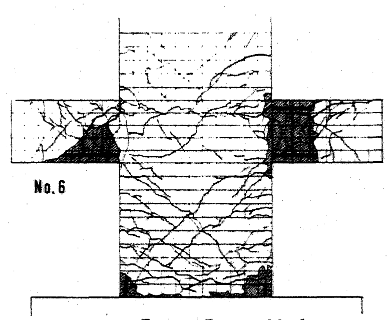


Fig.5 Failure Mode (Frame No.6)

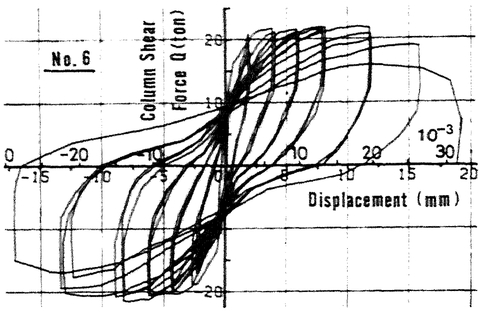


Fig.6 Hysteresis Curves (Frame No.6)

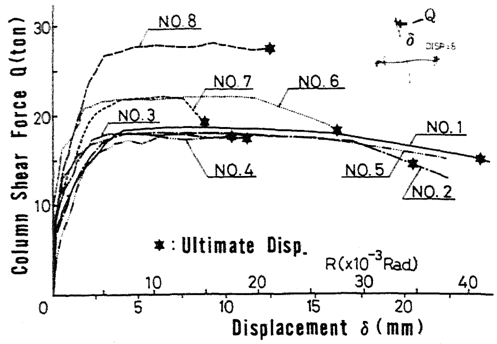
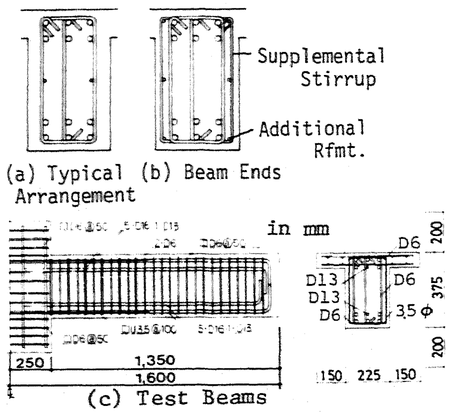


Fig.7 Envelopes of Hysteresis Curves



(c) Test Beams

Fig.8 Test Beams

Table 2 Test Beams

Beam No.	Main Bar	Stirrup (center)	Supplemental Stirrup
BO 1	1.55%	1.20% (0.85%)	-
BO 2	"	" "	-
BO 3	"	0.85 (0.85)	0.35%
BO 4	"	" "	0.085*
BO 5	"	" "	0.17*
BO 6	"	" "	0.35*
BO 7	"	0.42* (0.42*)	0.17*

· Shear Span Ratio : 2.8

* Ultra high strength bar σ_y > 13.0t/cm^2

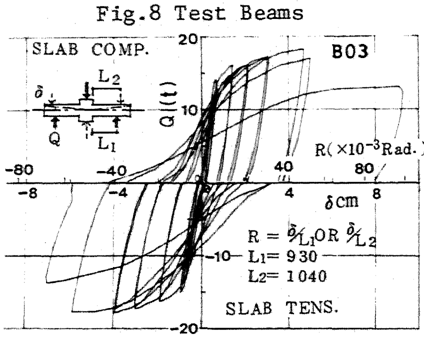


Fig.9 Hysteresis Curves (Beam B03)

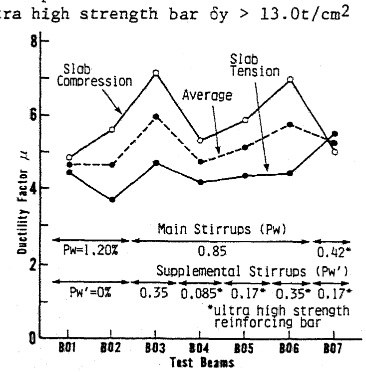


Fig.10 Ductility of Test Beams

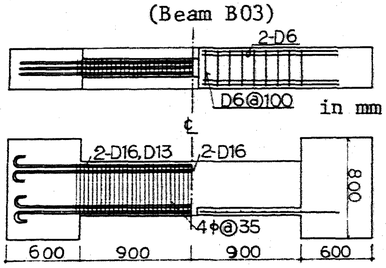
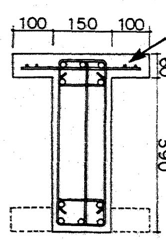


Fig.11 Test Exterior Cloumns



Longitudinal Rfmt. 1.37%

Transverse Rfmt. 0.72%

Column	M/Qd	Axial Stress
C1	2.0	-0.22fc' (tension)
C2	2.0	-0.22fc', 0.16fc'
C3	2.0, 4.7	-0.22fc', 0.53fc'

fc'=22.5MPa

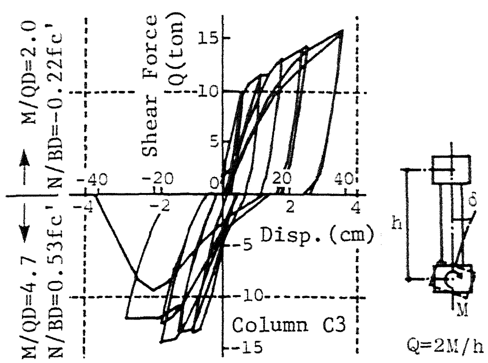


Fig.12 Hysteresis Curves
(Column C3)

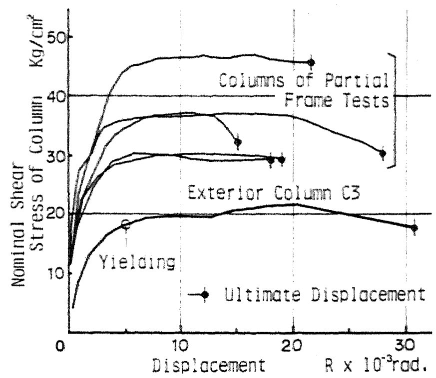


Fig.13 Envelops of Hysteresis Curves

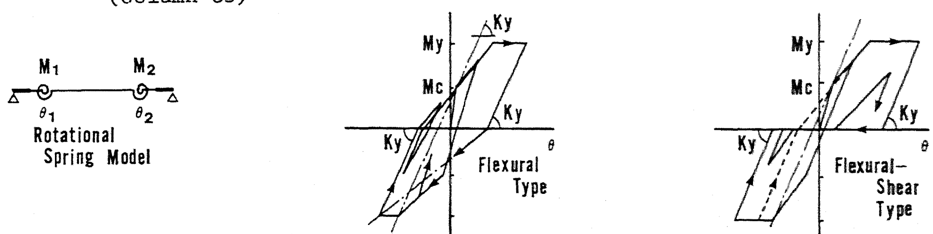


Fig.14 Beam Model and Hysteresis Rules for Nonlinear Static Analysis [2]

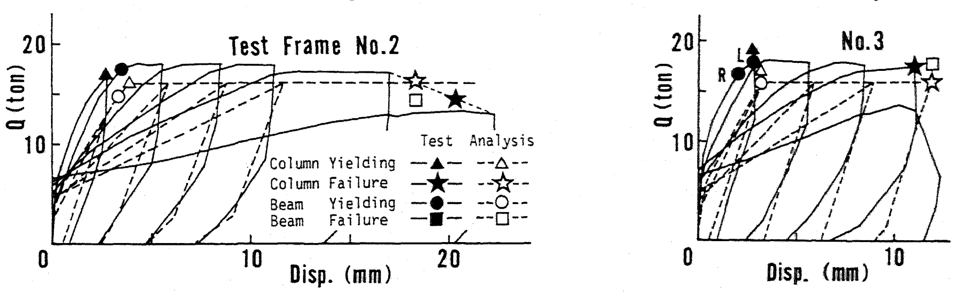


Fig.15 Nonlinear Analyses of Test Frames

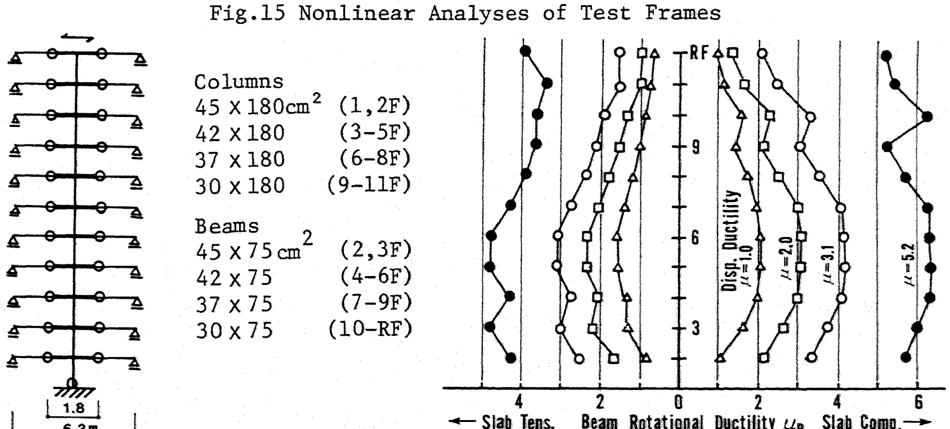


Fig.16 11-Story Model Frame and Analytical Ductility

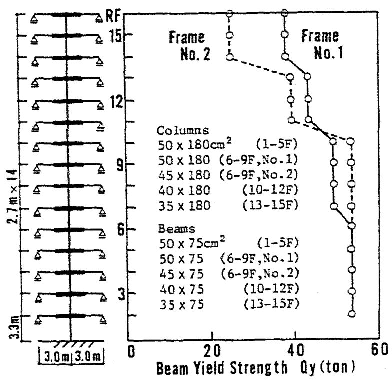


Fig.17 15-Story Model Frames for Nonlinear Analysis

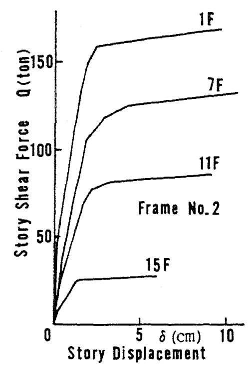


Fig.18 Load-Disp. Relation (Frame No.2)

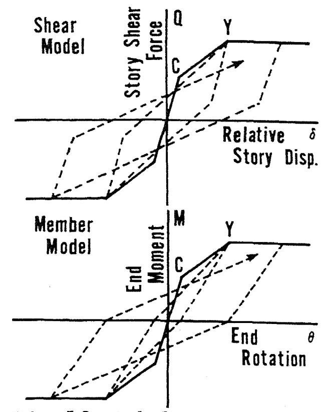
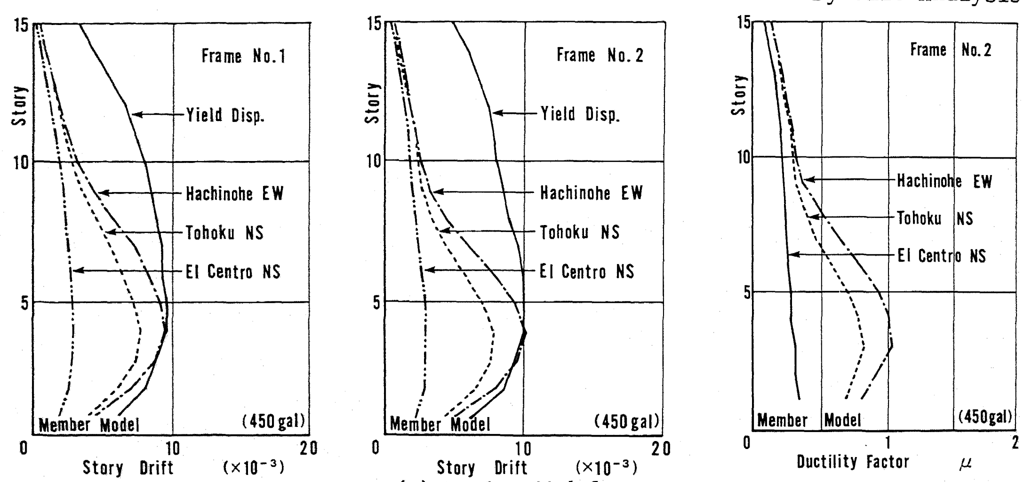
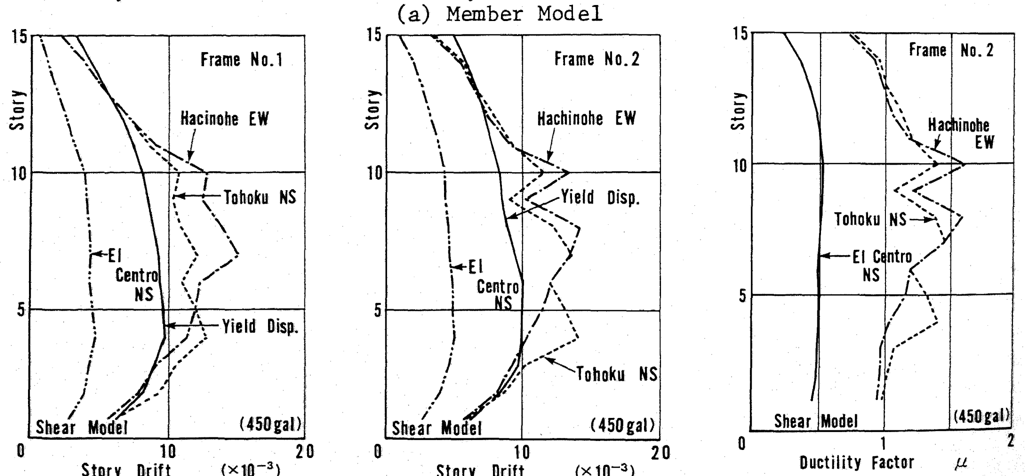


Fig.19 Models for Dynamic Analysis



(a) Member Model



(b) Shear Model

Fig.20 Dynamic Response Analysis of 15-Story Model Frames