U.S.-JAPAN COORDINATED EARTHQUAKE RESEARCH PROGRAM ON MASONRY BUILDINGS

-Seismic Test of Five Story Full Scale Reinforced Masonry Building, Part 2-

By

Shin Okamoto[1], Yutaka Yamazaki[2]

Takashi Kaminosono[3] and Masaomi Teshigawara[4]

SUMMARY

In order to investigate the validity of the design method and to know the total behavior of a RM building, static cyclic loading and pseudo dynamic tests of a five story full scale RM building was performed from mid-November, 1987 to the end of January, 1988. The test building in the extended service load range was very stiff with only minor structural distress. It was found that the characteristics of the test building in elastic range are very sensitive to the condition of foundation. The maximum lateral force capacity reached 968 ton, at an overall building drift of 4/800 (0.5%) in the yield load phase test.

Response of the pseudo dynamic test using the Taft EW 1952 (300gal) was within the hysteresis loop obtained from the yield load phase test. The ultimate deformation limit state was marked by rapid strength degradation at an overall building drift of 7/800 (0.875%).

The maximum shear carrying capacity and load-deformation relationship of the test structure was simulated well by a non-linear frame analysis.

KEY WORDS: Reinforced Masonry, Full Scale Test, Static Cyclic Load, Pseudo Dynamic Test, Non-Linear Frame Analysis

1. INTRODUCTION

Under the auspices of the UJNR on Wind and Seismic Effects, both the U.S. and Japan are working since 1984 on a coordinated earthquake research program on masonry structures. The target of the Japanese program is the development of comprehensive design guidelines for medium rise RM structures, in particular, the five story apartment building, to meet the countrie's need of high density residential construction. Based on a detailed understanding of the structural behavior of RM building derived from analytical and experimental research programs on materials, components, sub-assemblages and planar frame structures, the design guidelines (draft) was developed.

The five story full scale prototype test on a reinforced concrete masonry building was planned and carried out in order to experimentally verify analytical models which are assumed and dealt with in the draft of the design guidelines. The main portion of the full scale test consisted of three static cyclic load phases, those were service load phase, yield load phase, and ultimate load phase (see Table 1). The lateral load distribution was derived from the Japanese Building Code. Overall test plan, design and construction, and brief test results were presented at the last (20th) UJNR meeting held in Washington D.C. May 1988 [Ref.1].

In this paper, adding to the review of the test structure and loading method, results of each static cyclic load phase test and pseudo dynamic test carried out just before the ultimate load phase test are discussed. Pseudo dynamic test was performed in order to simulate the response of the damaged building due to the secondary shock after the main shock of an earthquake. Finally, large amplitude of deformation was applied to the test structure in order to obtain the deformation capacity of the test structure and the deformation characteristics of the structural elements in the ultimate state.

This paper also discusses on the reason why there is much difference between ultimate capacity calculated by the design guidelines and the one obtained from the test, as well as on the elastic behavior by mean of frame model analysis. A frame model analysis was applied because it is very simple and very convenient for current design method.

2. GEOMETRY AND LOADING OF TEST STRUCTURE

Even though detailed reference on the geometry and loading of the five story test building should be made to [Ref.1], a brief iteration of the most important geometrical data, load application and reference notation is presented in Fig.1 to allow quick cross references.

The five story full scale test specimen represented a module of a prototype apartment building with four parallel load bearing frames with openings (Frames Y1 through Y4, Fig. 1) in the critical loading direction under investigation. The floor area is 13.79m (loading direction) x 15.19m (transverse direction, including 2.4m wide balcony) and the total building height is 14m from the top of foundation to the roof slab (each story is 2.8m in height). The standard and corner concrete units used in the test building are shown in Fig.2. Running bond was used in the test building. Both vertical and horizontal rebars were arranged in 20cm spacing module basically. Wall length ratios are $15.20 cm/m^2$ in the loading direction, and $23.30 cm/m^2$ in the transverse direction.

[1] Dr. of Eng., Director, Research Planning & Information Department, Building Research Institute, Ministry of Construction, Japan.

[2] Dr. of Eng., Head, Production Department, Building Research Institute, Japan.

[3] Senior Research Engineer, Production Department, Building Research Institute, Japan.

[4] Dr. of Eng., Research Engineer, Production Department, Building Research Institute, Japan.

respectively. If the test building consists of 8 dwelling units per floor plan (4 of the test building modules side by side), those values are 14.89cm/m^2 and 18.15cm/m^2 in each direction, respectively.

Loads were applied to the center line (X2) of the test structure by means of 11 servo-controlled hydraulic actuators depicted schematically in Fig. 1(a). For the cyclic load testing, floor level loads were applied in forced control as a fraction of the roof level force as stipulated by the Japanese Building Code. Roof level loading was under displacement control to eliminate the torsional mode. For the pseudo dynamic seismic simulation test the five-story displacements were applied to one of the actuators at each floor level while the other servo-controlled actuator was force slaved to the first. Only at the roof level, where three actuators applied the lateral loading, see Fig.1(a), the two outside actuators were driven in displacement control again to eliminate the fundamental torsional mode.

Wall and beam/girder components were denominated by W or G, respectively, followed by a number and letter sequence indicating the plan location. Preceding digits 1 to 5 denote the story level under consideration. Thus, e.g., $_1W_{1A}$ denotes the first story wall labeled 1A at the lower left hand corner in Fig.1(b) at the intersection of reference lines Y1 and X1, etc.

3. SERVICE LOAD PHASE RESPONSE [Ref.2]

Crack pattern development in selected frames in loading direction is shown in Figs. 3 and 4. Base shear versus displacement relationship obtained through static cyclic loading tests is shown in Fig. 5 and corresponding roof level displacement envelope is shown in Fig. 6 together with indications of the major events detected during loading.

The service load phase covers the loading up to a nominal base shear stress level of 8.0kg/cm^2 which is approximately twice the nominal value corresponding to the 0.2G service load level required by the Japanese Building Code.

Up to the loading of 4.0kg/cm^2 nominal base shear stress level, the test structure showed very little distress and a very stiff overall response (The first story stiffness was 3,237ton/cm). Very small flexural cracks in the lintel beams and a slight opening of horizontal bed joint cracks in first and second story wall elements were observed at this loading stage. Also, cracking at slab corner started from this load level. The response of the test building up to this load level can be classified as virtually linear elastic.

During the loading between 4.0kg/cm^2 and 8.0kg/cm^2 nominal base shear stress level, flexural cracks at the horizontal bed joints of walls and the lintel beams in all stories developed, as shown in Fig. 3. Very small diagonal cracks starting from a corner of small rectangular openings located at the bottom of first and second story wall elements $_1W_{4A}$ and $_2W_{4A}$ started to develop. Cracks at floor slabs, perpendicular to the loading direction, started to open due to flexural deformation of beams concerned. Almost no cracks were observed in the orthogonal walls X_1 to X_3.

The first yield in any instrumented reinforcement was recorded in the flexural reinforcements of long wall elements $_1W_5$ and $_1W_8$, and in the building corner reinforcement in $_1W_{9B}$, all at 8.0kg/cm^2 nominal base shear stress level, as shown in Fig. 7.

The first story secant stiffness at this stress level decreased in 2,417ton/cm from initial elastic stiffness, 3,237ton/cm.

4. YIELD LOAD PHASE RESPONSE [Ref.2]

Crack Development and Reinforcement Yield Development

During the loading up to an overall building drift angle of 1/1,200 (0.083%), wall element $_1W_{4B}$ and a joint element located above the first story wall $_1W_{2B}$ exhibited diagonal cracking. At the loading of 9.0kg/cm^2 nominal base shear stress level, horizontal bed joint cracks of transverse wall elements were detected in the first story. Figure 7 shows flexural reinforcement yield development up to a building drift angle of 2/800 (0.25%). According to this figure, the reinforcements in all first story wall elements along the base of the building yielded up to a building drift angle of 2/800 (0.25%). The flexural reinforcement arranged at lower side of most beam elements also yielded until this deformation level. The flexural reinforcements arranged at upper side of all beam elements except $_2G_{7B}$ and $_2G_{6B}$ did not yield due to existence of RC floor slab.

At 2/800 (0.25%) drift angle load level, the first diagonal cracks occurred in most of all wall elements in the first story and also in the second story long wall element $_2W_8$. The stiffness of the building significantly dropped at this deformation level due to this diagonal cracking as seen in Fig. 6. The first diagonal cracking in a beam element also was detected in the second story short beam $_2G_{5B}$. Remarkable diagonal cracking from a corner of a small rectangular opening located at the bottom of the first story wall element $_1W_{4A}$ occurred, and radiated cracking in floor slabs adjacent to wall edges (including straight cracking along with a beam face) occurred as well at this deformation level.

At a deformation level of 3/800 (0.375%), diagonal cracks in first story wall elements increased in number. Face shell spalling in the second story short beam $_2G_{5B}$ as well as toe crushing of the long wall elements $_1W_5$ and $_1W_8$, which indicate the onset of major structural deterioration, started at this deformation level. In the first story orthogonal walls, vertical cracking was detected at the place distant 1.0-1.2 meters from the walls arranged in the loading direction, which is considered to be produced by excessive deflection of the walls in the loading direction. Lateral load still increased up to a deformation level of 4/800 (0.5%) drift angle at which the maximum lateral load (base shear) 968tons was recorded. This almost corresponds to the total building weight, 996tons.

Change in Stiffness and Deflection Mode

The two cycles service load level loading was held after the service load phase and the yield load phase responses in addition to the first service load level loading as shown in Table 1. Table 2 shows story stiffnesses of the test building under various loading level. It is clearly understood from this table that all the story stiffnesses changed at almost same rate of stiffness degradation. This also indicates that the building was considerably damaged under the yield load phase response up to a deformation level of 4/800 (0.5%) drift angle.

A plot of story drift percentage for each story, shown in Fig. 8, indicates that the deflection mode did not significantly change through cyclic loading up to the maximum lateral load level. One can just notice that the first story drift percentage is slightly increasing with increasing building drift.

Story rotation of the long wall element W_5 is depicted in Fig. 9. It is clearly noticed that flexural deflection is definitely caused by the rotation at the foot of the first story wall element ${}_1W_5$ and that, among three stories above the 3rd story, the sign of the story rotation is just opposite to the corresponding one in the first story, even the magnitude of story rotation at upper three stories is small. The fact mentioned in the latter suggests existence of considerable flexural reaction by beam elements in these stories. The shear and flexural deflection percentages in the total deflection of the first story wall element ${}_1W_5$ are 40% and 60% respectively through the building deformation level of the drift angles of 1/800 (0.125%)- 6/800 (0.75%).

The shear deflection versus story deflection relationship in the first story wall elements ${}_1W_{1B}$, ${}_1W_{2B}$ and ${}_1W_3$, depicted in Fig. 10, also clarifies that the shear deflection is, roughly speaking, 50% or a little less in the total story deflection up to the building deformation level of 4/800 (0.5%) drift angle.

Yield Hinge Development

The consecutive yield hinge development is depicted in Fig. 11. This almost coincides with reinforcement yield development as shown in Fig. 7. The difference between them is due to two kinds of difinition of yielding, that is, reinforcement yielding and structural element flexural yielding, as schematically shown in Fig. 12. The point 1 in Fig. 12 just corresponds to a reinforcement steel yielding point (2,000 micro strain in this case), whereas the point 2 was determined as a point where sudden change in slope on stress-strain relationship was observed, at which the structural element concerned is considered to have yielded.

Based upon Fig. 11, hinges were made at feet of most wall elements (65% of total number of hinges which were made finally) in the first story, and at second and third story short beams, ${}_2G_{5B}$ and ${}_3G_{5B}$, until a building drift angle reached 1/800 (0.125%). Among those hinges, the ones at feet of two first story long wall elements, ${}_1W_5$ and ${}_1W_8$, were first formed at 1/1,200 (0.083%) building drift angle. Up to 2/800 (0.25%) building drift angle, all the first story wall elements and some second story ones yielded at their feet. Beam elements, almost 60% of all, also yielded at their lower side throughout the entire building.

Up to 3/800 (0.375%) building drift angle, 64 - 66% of total number of hinges were made in wall and beam elements arranged in frames Y1 and Y4, whereas 80% in those arranged in frames Y2 and Y3 in which long wall elements were arranged.

During the loading up to 4/800 (0.5%) building drift angle, another yield hinges were newly made mostly at those elements arranged in frames Y1 and Y4 so that hinges more than 80% were formed in these frames until this drift angle. Most of all beam elements (83%) yielded at their lower side by then. Upper side of beam elements did not yield throughout the testing except second and fourth story beam elements in frame Y1 and two second story beam elements in frame Y4. This is considered to be caused by existence of RC floor slab reinforcements.

Strain Distribution in Reinforcing Bars

Many strain gages were attached to surface of reinforcing bars vertically and horizontally arranged in walls, beams and RC floor slabs to measure effectiveness of those bars especially arranged in the orthogonal elements. All the vertical reinforcements in the frame X_1 yielded at upper surface of building foundation(line H_1), when a building deformation level reached 1/1,200 (0.083%) drift angle, and also yielded at line H_2, 90 centimeters above the line H_1, under 1/800 (0.125%) drift angle loading. The first story vertical reinforcements in the interior frame X_2 yielded under relatively large deformation as compared with those in the exterior frame X_1.

All those reinforcements in the frames X_1 and X_2 yielded at floor slab surface seimaltaneously when flexural reinforcements arranged in concerned wall elements in the loading direction yielded. The yielding of vertical reinforcements arranged in the walls which are located perpendicular to the loading direction, thus clearly showed effectiveness of all those reinforcements against lateral loading.

Similar tendency was observed on floor slab reinforcement associated with beam flexural reinforcement. However, strain level through all the measurements of slab reinforcements were very low as compared with the one in vertical reinforcements arranged in orthogonal walls, such as the frame X_1.

5. PSEUDO DYNAMIC TEST [Ref.3]

The objective of the subsequent five-degree-of-freedom pseudo dynamic test was twofold, namely to investigate the response characteristics, i.e., natural frequency, mode of response and lateral force distribution, of the test structure after damage against 1.0G level lateral load accumulated, and to obtain the dynamic response of the damaged building to an after-shock or a subsequent seismic event with a 0.3G maximum acceleration.

The earthquake record of the Taft EW 1952 with a time window from 2.68 to 8.98 seconds was selected as the driving seismic motion for the pseudo dynamic test, since the first mode response was dominant with virtually equal amplitudes in both the positive and negative directions under preliminary computer analysis using the record.

The pseudo dynamic floor level response of the damaged test structure is depicted in Fig. 13 for the roof floor level displacement and base shear force time histories. After 6.3 seconds passed in pseudo dynamic response, the free vibration response produced a natural period of vibration of 0.54 second, slightly longer than the measured forced vibration response (0.41 sec.) after the yield load phase test.

Visual inspection of the test structure, i.e., trace of crack propagation and damage area assessment, did not indicate any additional structural damage or stiffness deterioration during the 300gals secondary seismic event. The obtained hysteretic response (Fig. 14) to the secondary seismic event stayed well within the hysteresis envelope obtained under the yield load phase test. Figure 14 again indicates that the test structure can still perform in a stable manner without increased loss of structural integrity.

Finally, Fig. 15 depicts the variation of peak response of the test structure along the height. The dominance of the first mode response is clearly visible as well as the limiting response envelope obtained at the ultimate lateral load limit state during the cyclic testing of the yield load phase. Of particular interest is the shape of the lateral load distribution over the building height, Fig. 15a, which is almost linear (inverse triangular) as opposed to the shape of the imposed lateral force distribution for the cyclic load testing which was based on the Japanese Building Code. The straight line inverse triangular response is indicative of the formation of flexural mechanisms at the base of the first story wall elements. This tendency is emphasized by almost straight line lateral displacement distributions along the building height, see Fig. 15b, compared to the displacement envelopes obtained during the cyclic yield load phase test.

6. ULTIMATE LOAD PHASE RESPONSE [Ref.3]

Subsequent to the maximum lateral load limit test and the pseudo dynamic seismic test, the test structure was subjected to increased cyclic lateral deformation levels, beyond the 4/800 (0.5%) building drift at maximum load, to investigate the strength degradation characteristics and to evaluate the ultimate failure mechanism. Overall building drift levels of 5/800 (0.625%), 6/800 (0.75%) and 7/800 (0.875%) were imposed in single load cycle with full reversal. At a total building drift of 7/800 (0.875%) the ultimate deformation load test was terminated since a lateral load carrying capacity of less than 50% of the maximum lateral load level was obtained on the reverse deformation cycle.

The base shear versus drift relationship for the overall building (roof level) and the first story level are depicted in Fig. 5. In the initial deformation cycle load levels of 98%, 90% and 69% of the maximum lateral load were achieved at overall building drift levels of 5/800 (0.625%), 6/800 (0.75%) and the 7/800 (0.875%), respectively. A comparison between the overall building drift and the first story drift shows first story drift levels of 1/102 (0.98%), 1/70 (1.43%) and 1/42 (2.38%) for the corresponding overall building drift levels of 5/800 (0.625%), 6/800 (0.75%) and 7/800 (0.875%). The cyclic ultimate deformation load test showed that 80% of the maximum lateral load capacity was still maintained at overall building drift of 1/120 (0.83%) and first story drift of 1/60 (1.67%).

The lateral deformation characteristics of the test building over the building height is depicted in Figs. 16 and 17. Individual story displacements and story drifts are plotted over the height of the test structure for various overall building drift levels in Figs. 16a and 16b, respectively. A significant increase in first story deformation and drift for overall building drift levels of 6/800 (0.75%) and 7/800 (0.875%) can be seen, with very little additional deformations in the upper stories two to five. The rapid increase in story drift without an increased rotational component is a clear sign for shear deformations and the development of a shear mechanism in the first story walls. This tendency is also depicted by the overall deformation modes for lateral load bearing frame Y2 in Fig. 17, where deformation states at 3/800 (0.375%) and 7/800 (0.875%) radians are compared. The additional upper story deformations are negligible compared with the first story displacements.

The above findings are confirmed by large diagonal cracks in all first story walls as depicted by the ultimate crack pattern of all four load bearing frames in Fig. 18. The opening of these diagonal cracks in the first story wall components occurred at deformation levels of 5/800 (0.625%) and 6/800 (0.75%) building drift, which corresponds to the increased lateral deformations in the first story at these load levels. Few of the cracks in the upper stories as well as the cracks in the floor slabs and in the transverse walls extended or widened during the ultimate deformation load phase test. Thus, the overall deformation mechanism encountered in the test structure clearly shifted from an overall flexural behavior up to and at the ultimate load limit state (5/800 (0.625%) building drift) to a dominated mechanism in the first story during the ultimate deformation load phase test.

7. FRAME ANALYSIS [Refs. 4 and 5]

Model

The test building is idealized to be frames composed of beam members representing the wall and beam components. In order to represent the nonlinearity of members, walls and beams, one-component model consisting of flexural spring at the both edges of a member and a shear spring at the center of a member is considered. Beam model has rigid zones at both ends. Flexural spring characteristics representing flexural behavior and shear spring characteristics representing shear behavior of a member are modeled to be a tri-linear one with cracking and yielding points.

For flexural spring, yield rotation angle(τ_y) is assumed to be 1/800 for 1m long wall and beam, 1/1,500 for 2m long wall and 1/3,000 for 4m long wall. Maximum flexural strength is calculated by the equation in Ref. 7. Those values are listed in Table 3. For shear spring, nominal shear cracking stress is assumed to be $15kg/cm^2$. Therefore shear cracking deformation angle(γ_c) becomes almost 1/4,000.

The value of Young's Modulus (E) is defined from the prism compressive test results. In this analysis mean values of E and F_m are $196t/cm^2$ and $196kg/cm^2$, respectively. Shear Modulus (G) is gotten assuming that poisson's ratio(ν) is 0.2 ($G=86t/cm^2$).

The joint part of wall and beam is treated as rigid zone in this analysis. In the elastic analysis this rigid zone is assumed as illustrated in Fig. 19 and in the inelastic analysis full part of beam-wall joint is assumed to be rigid zone. The effective width of transverse members for elastic stiffness are determined by Ref. 7.

Typical component and subassemblage test can be traced with this beam model and the validity of an inelastic beam (member) model used in this paper is verified [Refs. 4 and 5].

Foundation

Test building is fixed to testing floor (depth = 2.00m). Test building is too rigid to consider the foundation be fixed. In this analysis foundation beam ($I=121.67x10^5cm^4$, $A=10,575cm^2$) and testing floor ($I=600x10^5cm^4$, $A=18,000cm^2$) and the rotational stiffness (see Table 3 c) due to prestressing bars are considered.

Results of Elastic Stage

Four cases listed in Table 4 are analized. Among Model 1 through Model 3, different effective width of transverse members to inertia moment is considered. In Model 1, rectangular section without taking account of transverse members is considered. In Model 2, the full width of transverse members are taken into each member's inertia moment. The inertia moment of **Model 3 is** defined according to the design guidelines [Ref.7]. The design guidelines recommend that incremental factor of inertia moment due to transverse members is up to 2.0, and inertia moment of almost all members with transverse members are limited to be 2.0 times as that of rectangular section (I_o). These three cases, Model 1 through Model 3, are assumed to be under fixed end condition. In Model 4 the effect of the foundation is considered, other conditions besides the foundation are the same as corresponding ones in the Model 3.

The vibration periods are tabulated in Table 5. Experimental results are also listed in this table. Experimental results are obtained from the flexural matrix gotten by the each floor unit load test. Model 4 gives the closest results to the test ones among these Models. The limitation of effective inertia moment up to $2*I_o$, is reasonable. The recommendation for effective width of transverse members in the design guidelines is also reasonable. Vibration modes between Model 3 and Model 4, are different each other. It clearly shows the necessity to consider the effect of foundation condition in elastic stage.

Overall Behavior in Inelastic Stage

Monotonic loading analysis is carried out. Inertia moment and foundation condition are the same as the Model 4. In this case, however, full part of the panel zone surrounded with walls and beams is taken into account as a rigid zone. This is based on the fact that the purpose of this analysis is to predict the lateral load capacity of the test building from the component strength at their critical section, and also that the stiffness degradation of each member is so dominant to the total stiffness of the test building that rigid zone of wall-beam panel is assumed not to affect on the total stiffness of the test structure. Maximum moment capacity of each member is calculated with considering the effect of all re-bars in transverse members.

Figure 20 shows the each story shear force vs. story drift angle relationship. Good agreements between test results and this analysis are observed. This means that the effect of re-bars in transverse member on flexural strength and characteristics of flexural and shear springs are evaluated properly in this analysis. These spring characteristics are defined based upon the component test results conducted in past years.

Development of Yield Hinges

Test result shows that the yielding of wall W_5 occurred at first in the nominal shear stress of $8kg/cm^2$ loading step. But it means that the extreme edge re-bar reached its yield strain at this loading level. The re-bars' strains in middle part of wall W_5 increased rapidly from building drift angles of 1/800-2/800 (0.125%-0.25%). This fact matches well to the analytical results.

8. CONCLUSIONS

Conclusions obtained from the seismic test on the five story full scale reinforced masonry building are listed as follows:

Service and Yield Load Phase Tests

1. The test building showed very little distress and a very stiff overall response against the loading of $4.0kg/cm^2$ nominal base shear stress level which corresponds to the 0.2G design load level. The first story drift angle of the test building against the 0.2G design load level was 1/4,530. Almost no crack was observed under this design load level loading. The test building certainly exceeded design requirements of carrying lateral load capacity corresponding to 0.5G base shear which is specified in a draft of a seismic design guidelines for medium rise RM buildings.

2. The actual lateral load capacity obtained during the test at an overall building drift angle of 4/800 (0.5%) was 968tons which is almost equivalent to the weight of the test building (dead load + live load) or a base shear coefficient of 1.0G.

3. The overall development of failure mechanisms was governed by formation of flexural yield hinges at the base of the first story walls, especially walls W_3, W_5 and W_8, and also at the beam ends throughout the entire building. All first story walls clearly yielded in flexure long before the development of major diagonal cracks which ultimately dominated the first story wall limit behavior.

4. Significant portions of the transverse walls contributed to the overall load transfer in the form of wide flanges to the four load bearing frames. Activation of all the transverse wall reinforcement certainly contributed to the high maximum lateral load capacity. Large contribution of floor slab to beam strength can be considered from the observed transverse crack patterns in the reinforced concrete floors and also from strain distribution in the floor slab reinforcements associated with flexural reinforcements in the beams.

Pseudo Dynamic Test

5. In spite of the comparatively large magnitude of the input acceleration, 300 gals, response of the specimen was within the hysteresis loop obtained from the yield load phase test, and there was no progress in damage of the specimen. The lateral force distribution was more similar to inverted triangular distribution than that imposed in the static loading test, and also the deflection mode was nearly linear compared with the one in the yield load phase test. The natural period of 0.54sec. was obtained from the free vibration by means of the pseudo dynamic test method.

Ultimate Load Phase Test

6. Over 5/800 (0.625%) building drift angle, lateral load capacity decreased gradually and deflection started to concentrate to the first story. Flexural deflection mode was changed to shear failure mode which was dominated by only the first story displacement. After 6/800 (0.75%) building drift angle, shear cracks at the first story walls expanded and lateral load capacity of the test building decreased rapidly with concentration of deflection to the first story. The overall building drift angle was approximately 1/120 (0.83%) when the strength decreased to 80% of the maximum strength.

Frame Analysis

7. Total behavior of the test building was simulated by a non-linear frame analysis. In this analysis, wall and beam characteristics are assumed based on a component test and simple beam theory. The elastic analysis showed that rigid zone recommended in the RM design guidelines is reasonable. And the ultimate analysis also showed that the lateral load capacity of the test building can be estimated by the flexural strength at critical section of each component. It is also important for discussing the elastic behavior of such a rigid structure to consider the condition of foundation. And in inelastic stage, all the reinforcing bars in the transverse members are effective on the ultimate shear carrying capacity.

ACKNOWLEDGMENTS

The five story full scale masonry test was carried out as a part of the U.S.-Japan coordinated earthquake research program under the auspices of the UJNR on Wind and Seismic Effects. In Japan side, the program was promoted under the Promotion Committee for Masonry Research (PROCMAR, chaired by Prof. H. Umemura), the Technical Coordinating Committee for Masonry Research (TECCMAR, chaired by Prof. T. Okada) and the Building Construction Committee for Masonry Research (BLDCMAR, chaired by Prof. K. Kamimura). The writers would like to express their gratitude to these committees. The contributions of the U.S.-TCCMAR members, especially Prof. F. Seible, UCSD, in numerous discussions and joint meetings also are greatly appreciated.

The writers would like to record the generous cooperation of Dr. H. Mizuno, Messrs H. Kato, H. Isoishi, T. Nishi, and M. Matsuno for performing the full-scale RM building test.

REFERENCES

[1] S. Okamoto, Y. Yamazaki, T. Kaminosono, and M. Teshigawara, "Seismic Test of the Five Story Full-Scale Reinforced Masonry Building", 20th Panel Meeting on Wind and Seismic Effects, UJNR Cooperative Research Program, Washington D.C., May 17-20, 1988.

[2] Y. Yamazaki, et al., "Seismic Test of Five

story Full Scale Reinforced Masonry Building-Outline of Test and, Service and Yield Phase Response", Proceedings of the Fourth Meeting of the U.S.-Japan Joint Technical Coordinating Committee on Masonry Research, San Diego, October, 1988.

[3] T. Kaminosono, et al., "Seismic Test of the Five-Story Full-Scale Reinforced Masonry Building-Pseudo Dynamic Test and Ultimate Phase Response-, "Proceedings of the Fourth Meeting of the U.S.-Japan Joint Technical Coordinating Committee on Masonry Research, San Diego, October 1988.

[4] M. Teshigawara, "Introduction of Dynamic Inelastic Frame Analysis on Reinforced Masonry Structures," Proceedings of the Second U.S.-Japan Joint Technical Coordinating Committee Meeting on Masonry Research, Keystone, USA, 1986.

[5] M. Teshigawara, T. Kaminosono, Y. Yamazaki and T. Nishi," Seismic Test of Five Story Full Scale Reinforced Masonry Building- Analyses on Service and Yield Phase Response-", Proceedings of the Fourth Meeting of the U.S.-Japan Joint Technical Coordinating Committee on Masonry Research, San Diego, October, 1988.

[6] "Design Standard for Reinforced Concrete Building," Architectural Institute of Japan.

[7] T. Kaminosono, et al., "Draft of Design Guidelines for Reinforced Masonry Buildings," Proceedings of the Third Meeting of the U.S.-Japan Joint Technical Coordinating Committee on Masonry Research, Tomamu, Hokkaido, Japan, October 1987.

Table 1 Test and Loading Sequence

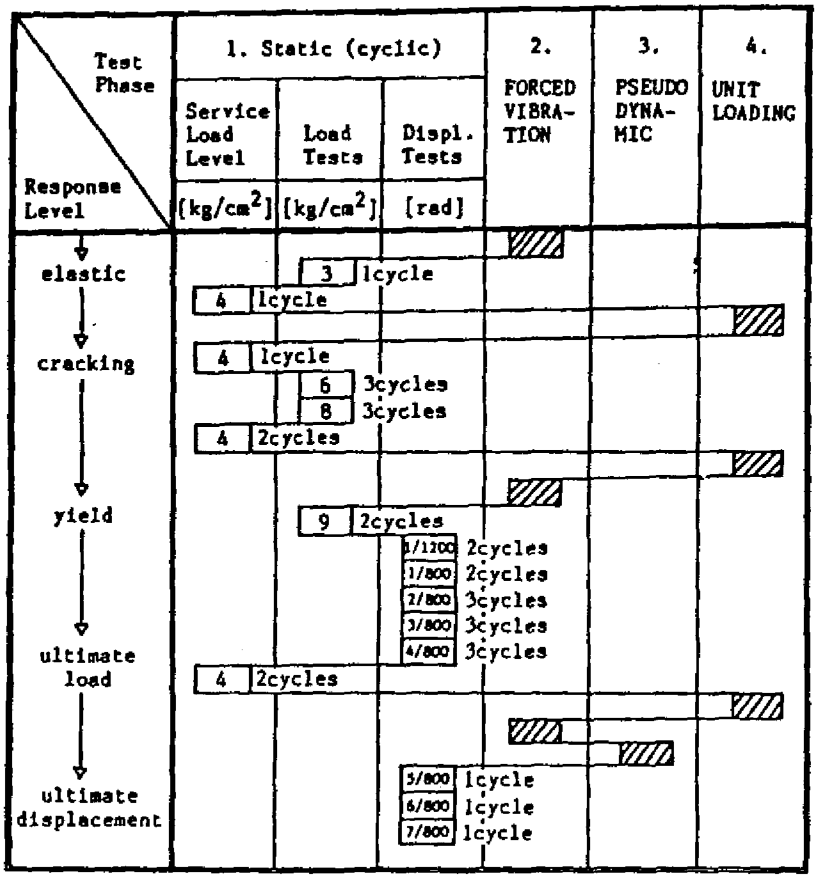


Table 2 Story Stiffness of 5 Story Full Scale Test Building

[ton/cm]

story	$4kg/cm^2$ Stress Level			$8kg/cm^2$ Stress Level	1/800 building Drift Angle Level
	Virginal Loading	After Service Load Phase	After Yield Load Phase		
5	1975 (100)	1660 (84.2)	230 (11.6)	1641 (83.1)	844 (42.7)
4	2520 (100)	1906 (75.6)	340 (13.5)	1978 (78.5)	1240 (49.2)
3	2763 (100)	2158 (78.1)	363 (13.1)	2180 (78.9)	1329 (48.1)
2	3237 (100)	2401 (74.2)	394 (12.2)	2464 (76.1)	1525 (47.1)
1	3237 (100)	2392 (73.9)	291 (9.0)	2417 (74.7)	1554 (48.0)

Table 3 a) Characteristics of Member-Beam-

Member	My_1	My_2
$_RG_3$	889	9,150
$_5G_3$	1,228	9,589
$_4G_3$	1,355	9,940
$_{3,2}G_3$	1,566	9,940

My_1: Ultimate moment when lower side is in tention [t.cm]
My_2: Ultimate moment when upper side is in tention [t.cm], including the re-bars in slab (28D10 and 8D13)
Mc : Flexural cracking moment = My/3
$I = 15.6*10^5 cm^2$
$I < 2*I_o$, $I_o = bD^3/12$ $(7.8*10^5 cm^4)$
$\gamma_y = 1/800$ rad.
A : Area for shear stiffness = $1{,}501 cm^2*1.2$, 1.2:effect of RC slab
Qc= Strength of shear cracking = $15 kg/cm^2*A$ = 27.0ton
Foundation Beam. $I=721.67*10^5 cm$, $A=28{,}575 cm^2$

Table 3 b) Characteristics of Member-Wall-

Member		My	I,Qc, y,A
W_{4A}	5	10,544	
	4	8,178	$I=249.55*10^5 cm^4$
	3	8,781	Qc=68.0 ton
	2	6,784	$\gamma_y=1/1{,}500$
	1	4,192	$A=3{,}781 cm^2$
W_5	5	31,301	
	4	34,997	$I=1164.0*10^5$
	3	43,395	Qc=143.3
	2	47,091	$\gamma_y=1/3{,}000$
	1	50,787	A=7961
W_{4B}	5	7,559	
	4	11,555	$I=249.55*10^5$
	3	16,200	Qc=68.0
	2	20,510	$\gamma_y=1/1{,}500$
	1	21,058	A=3,781

My : Ultimate moment [t.cm]
$Ts = 15 kg/cm^2$
Qc = Ts*A*1.2. 1.2 : Effect of transverse wall
A : Area for shear stiffness and strength

Table 3 c) Characteristics of Rocking

Location	K_T(t.cm)
2m length wall	$420*10^4$
4m length wall	$1,700*10^4$

K_T=2* 32*Es*D^2/l=105D^2
Es=2,100t/cm_2 32=8cm_2 l:length of PC-bar(320cm)
D:wall length

Table 4. Analytical Condition of Model 1-4

	Model 1	Model 2	Model 3	Model 4
Transverse member	Not considered	Full width considered	According to the RM D.G[7]	Same as the Model 3
Foundation	Not considered (Fix)	Not considered (Fix)	Not considered (Fix)	Considered
Shear area	Rectangular only			

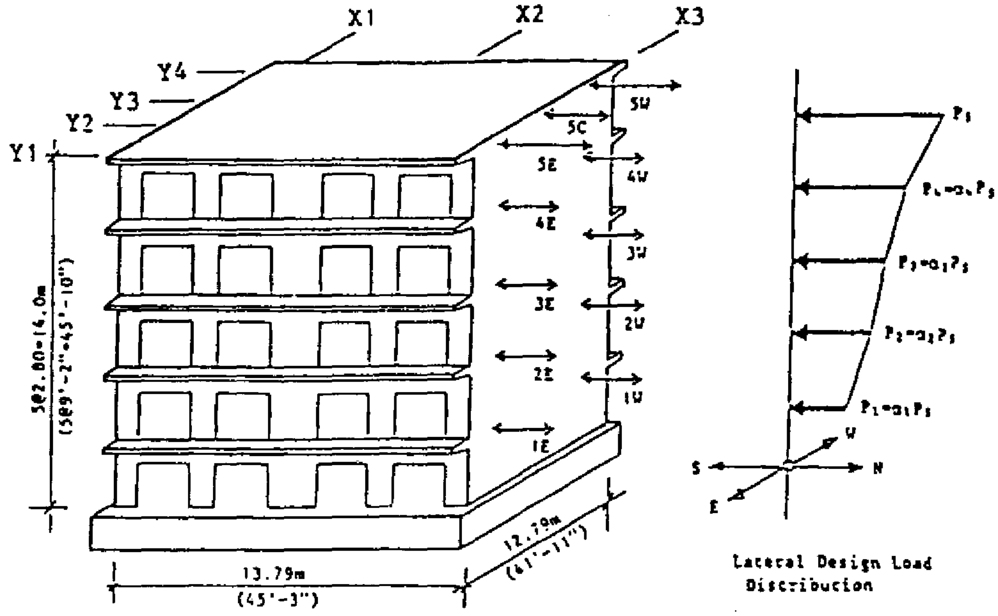
E = 196 ton/cm^2 G = 86 ton/cm^2

In the elastic analysis for Model-1 to Model-4, rigid zone of wall-beam joint is assumed as shown in Fig. 19 (according to A.I.J. standard for RC structure [Ref.6]).

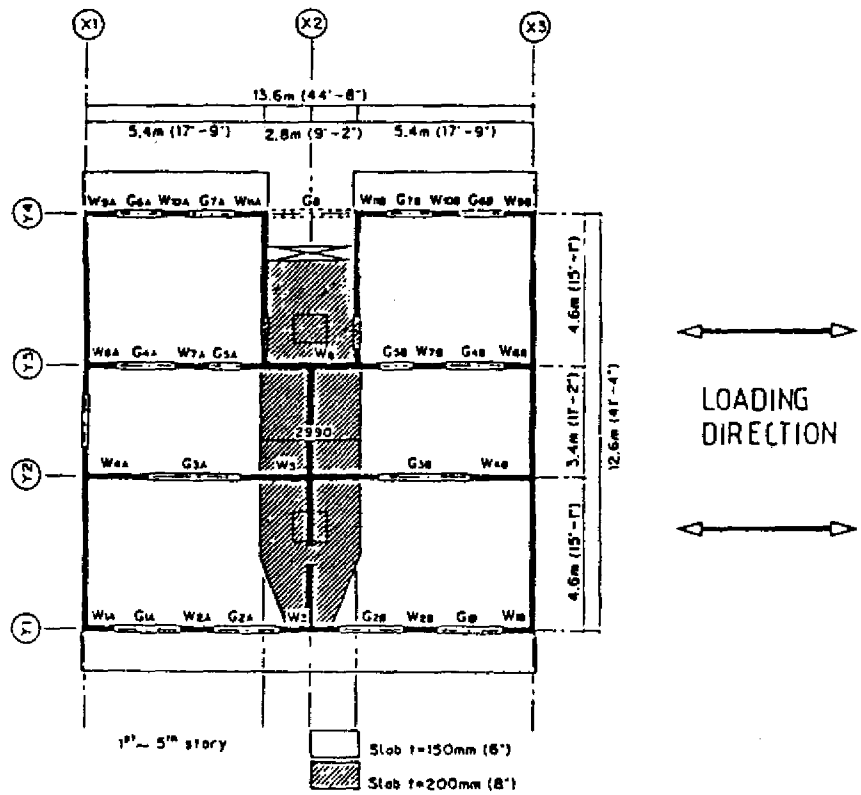
Table 5 Natural Period (sec.)

	Model					
MODE	M 1	M 2	M 3	M 4	M 5	TEST
1	0.184	0.12	0.159	0.169	0.166	0.157
2	0.055	0.039	0.048	0.052	0.051	0.049
3	0.029	0.022	0.026	0.028	0.027	0.026
4	0.020	0.016	0.018	0.019	0.019	0.019
5	0.016	0.013	0.015	0.015	0.016	0.013
	FIX END CONDITION					

* note: It is very difficult to evaluate the proper rotational stiffness of foundation because of its pre-stressed PC bars. Model 4 ignores the effect of this pre-stressed. In supplementary analysis, Model 5, rotational spring due to pre-stressed PC bars was changed to be 1,000 times rigid as compared with Model 4. Natural period does not changed so much(T1=0.169 sec in Model 4, T1=0.166 sec. in Model 5). And the first vibration mode of Model 5 closes to the test results than that of Model 4. It is supposed that rotational stiffness assumed in Model 4 would be enough rigid.



a) Test Setup and Load Distribution



b) Plan Geometry and Reference Notation

Fig.1 Geometry and Loading of Test Structure

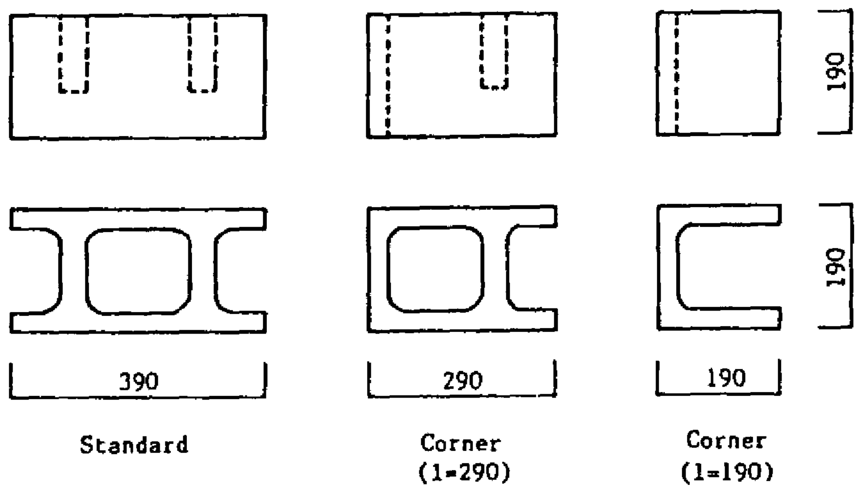


Fig.2 Concrete Block Units [scale in mm]

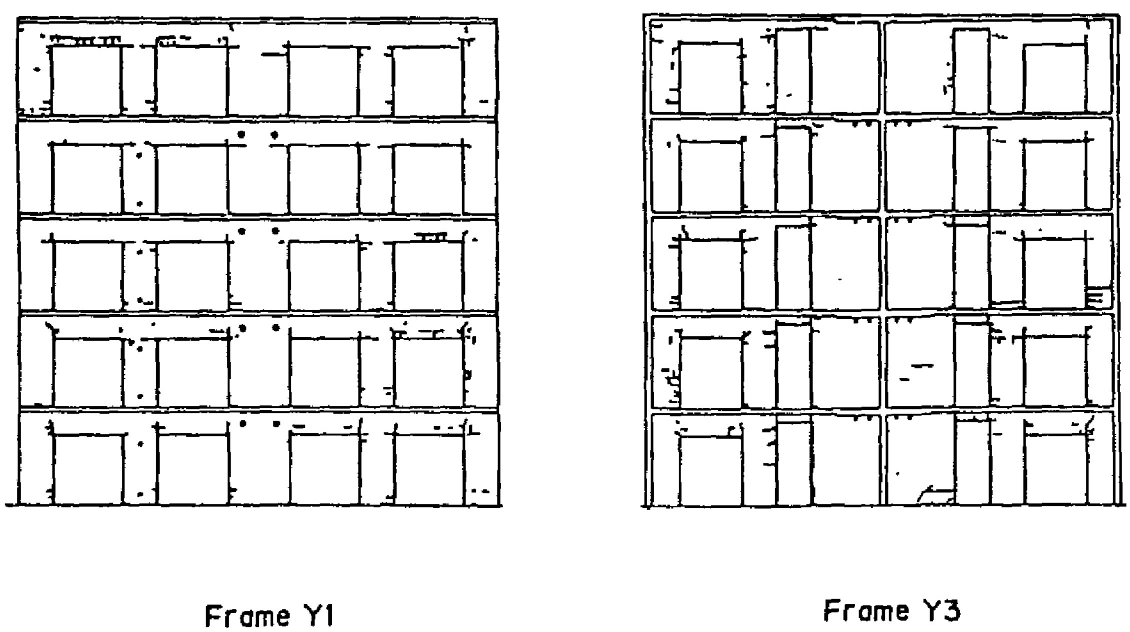
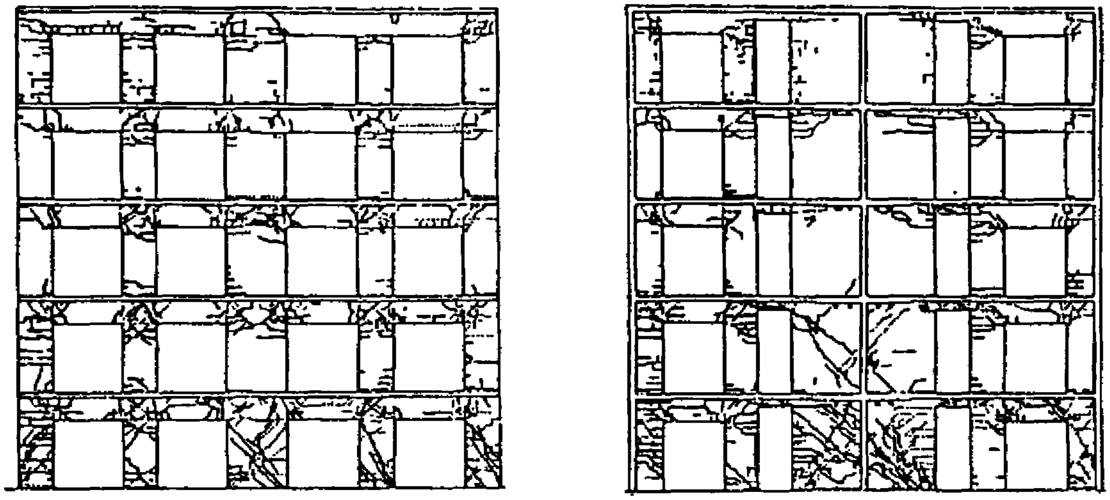
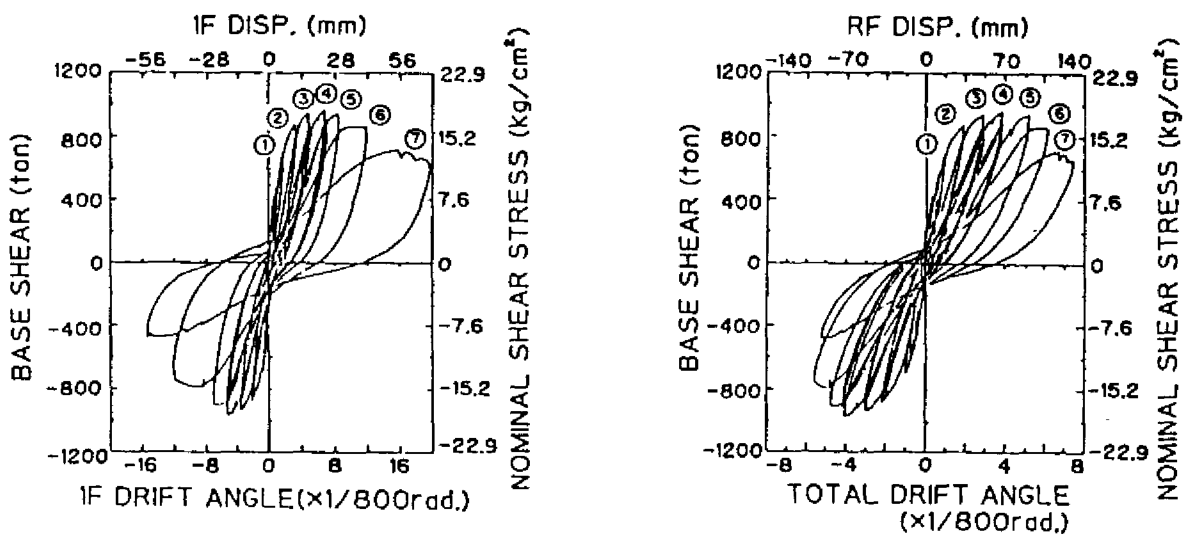


Fig.3 Crack Pattern of Frames -Y1 and -Y3 at τ=8kg/cm^2



Frame Y1 Frame Y3

Fig.4 Crack Pattern of Frames -Y1 and -Y3 at R_t=4/800 rad.



b) 1st Story Drift

a) Building Drift

Overall Building Drift Levels

1. R = 1/800rad. or 0.125%
2. R = 2/800rad. or 0.250%
3. R = 3/800rad. or 0.375%
4. R = 4/800rad. or 0.500%
5. R = 5/800rad. or 0.625%
6. R = 6/800rad. or 0.750%
7. R = 7/800rad. or 0.875%

Fig.5 Base Shear vs. Displacement Relationship
Obtained through Static Cyclic Loading Tests

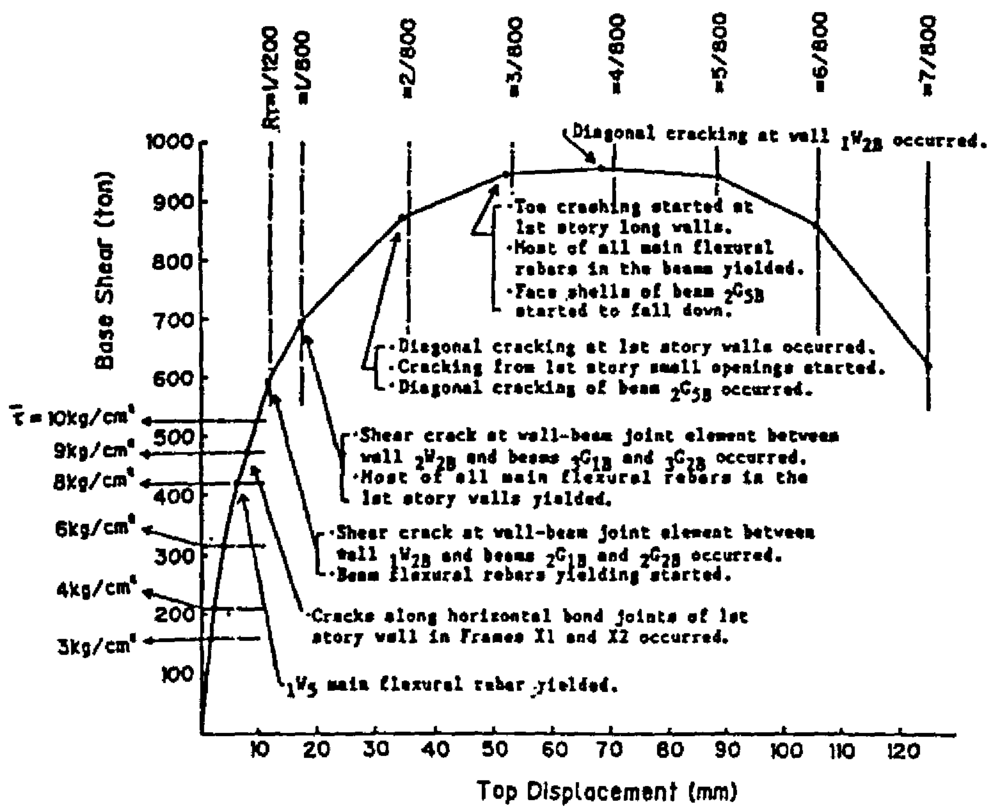
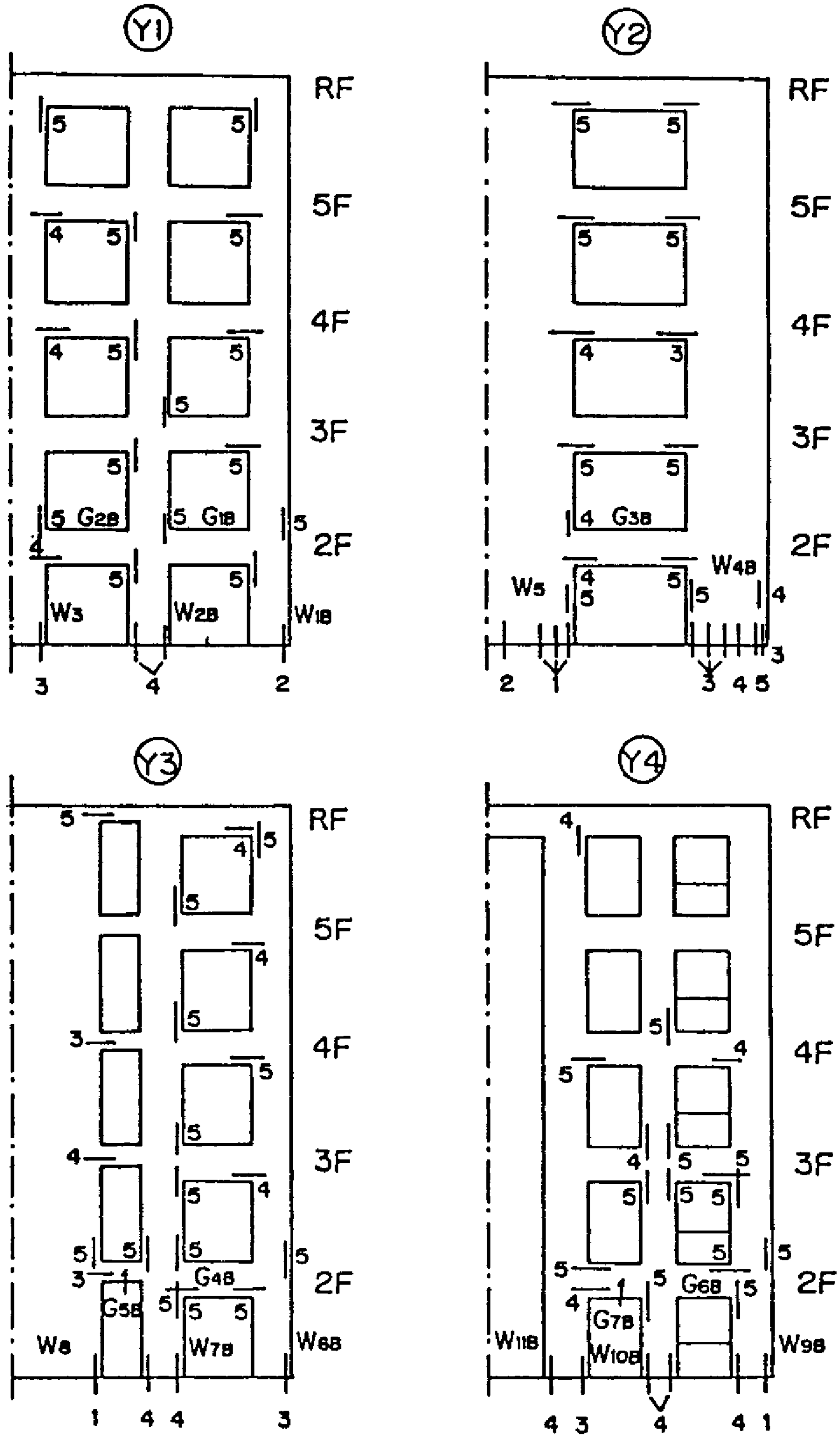


Fig.6 Base Shear vs. Top Displacement Envelop
Curve for Static Cyclic Loading Tests



Building Drift	2 -1/1670(0.060%)	4 -1/800(0.125%)
1 -1/2000(0.050%)	3 -1/1200(0.083%)	5 -2/800(0.250%)

Fig.7 Consecutive Reinforcement Yield Development

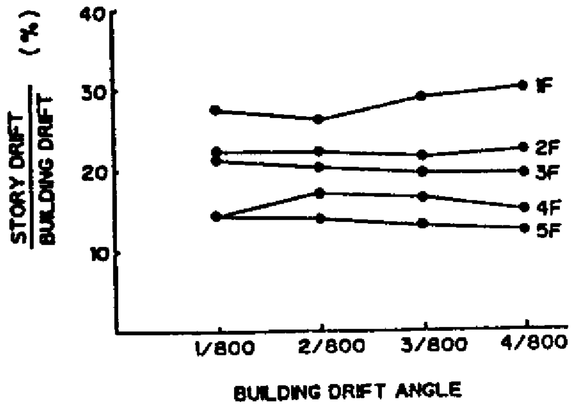


Fig.8 Story Drift Percentage during
Yield Load Phase Pesponse

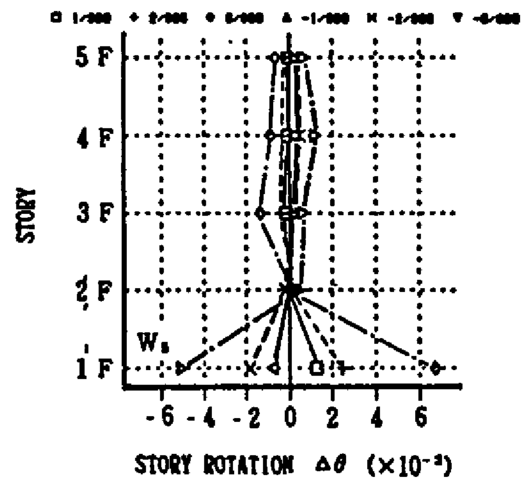


Fig.9 Story Rotation of Wall W5

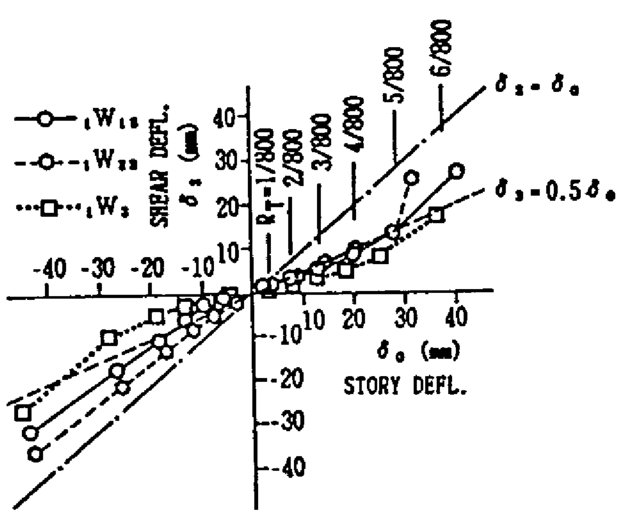


Fig.10 Shear Deflection vs. Story Deflection
on First Story Walls

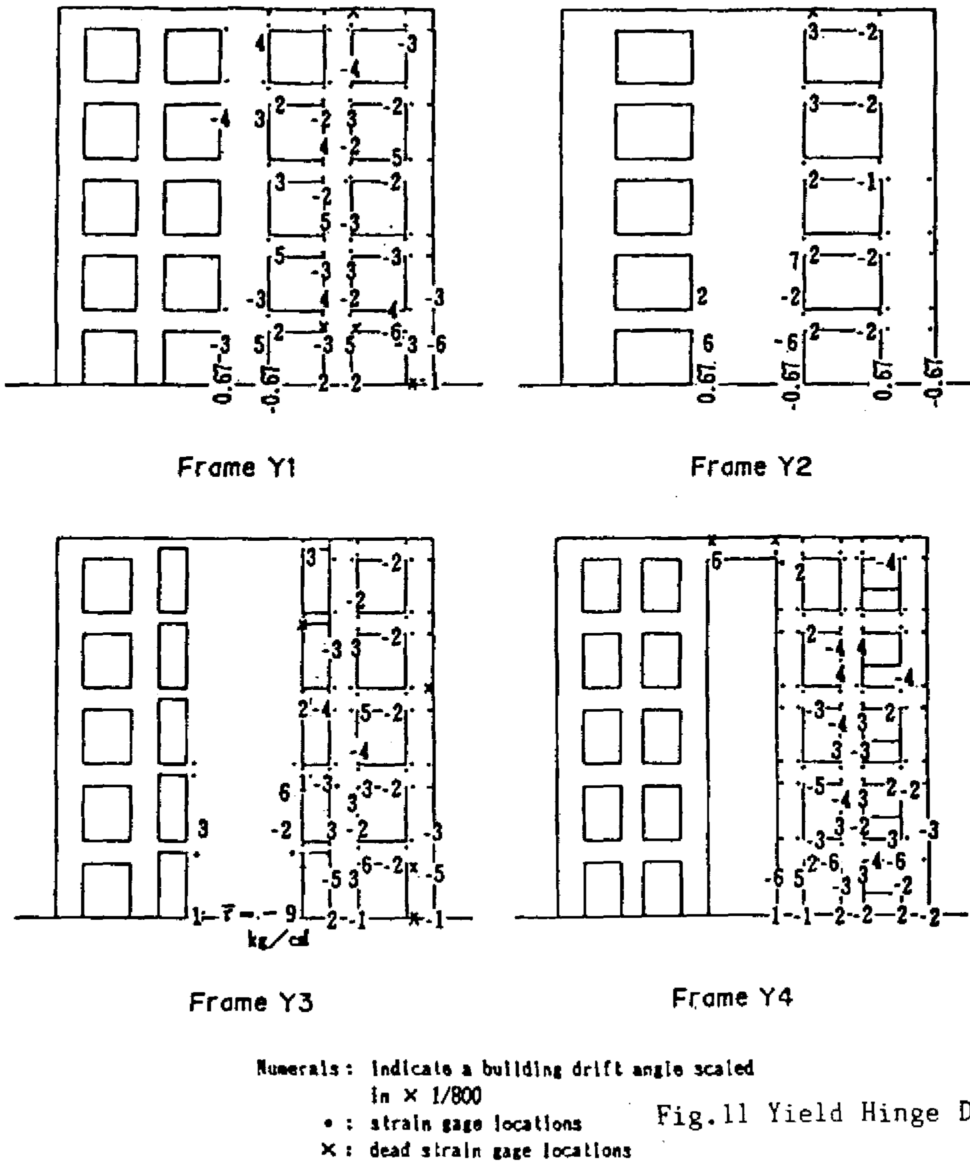


Fig.11 Yield Hinge Development

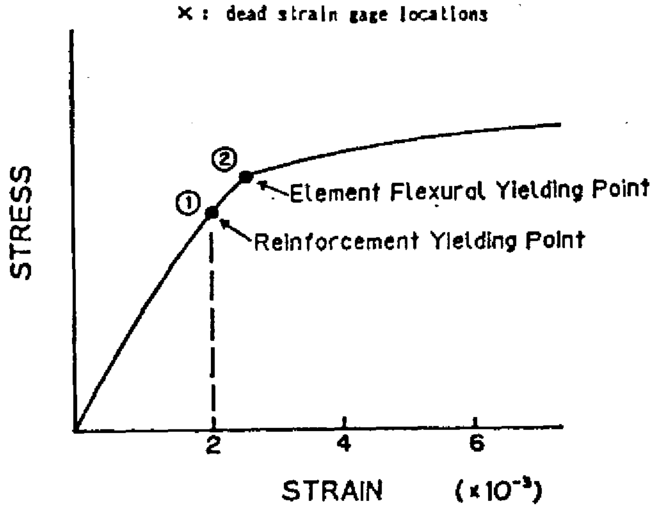


Fig.12 Difinition of Yielding

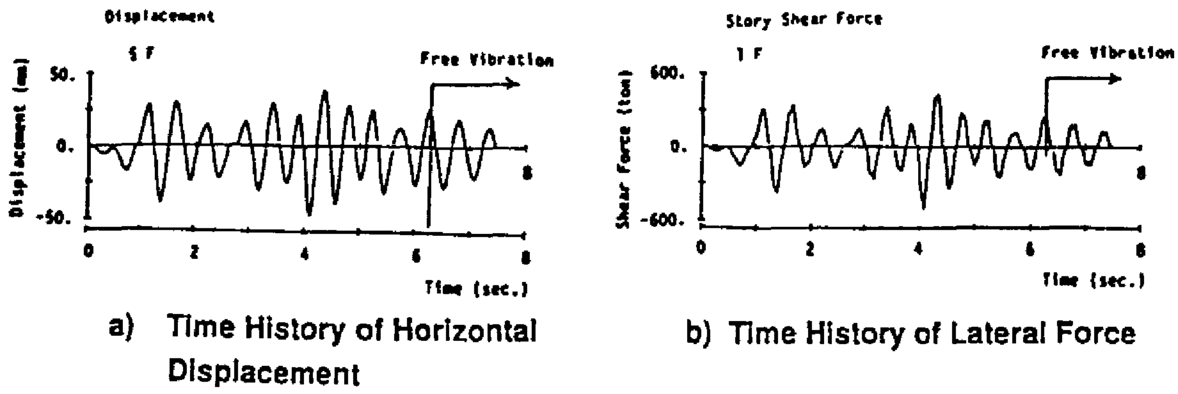


Fig.13 Pseudo Dynamic Test Floor Level Response to Taft EW 1952 (0.3g)

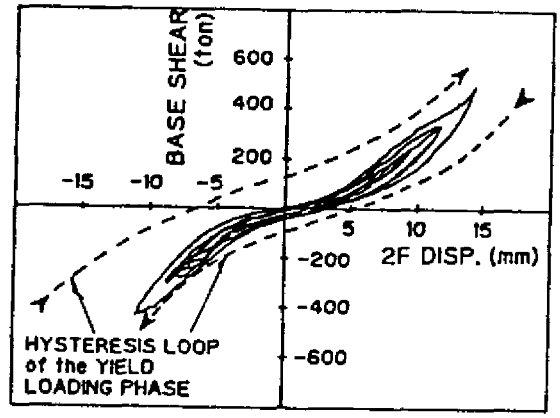


Fig.14 Relationship Between Base Shear and the First Story Displacement During Pseudo Dynamic Test

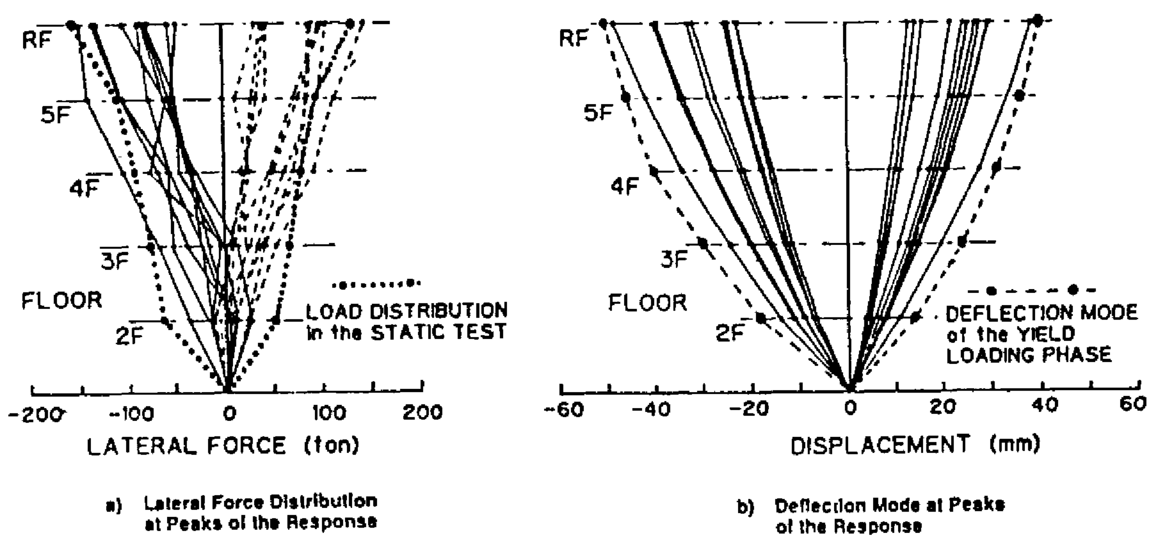
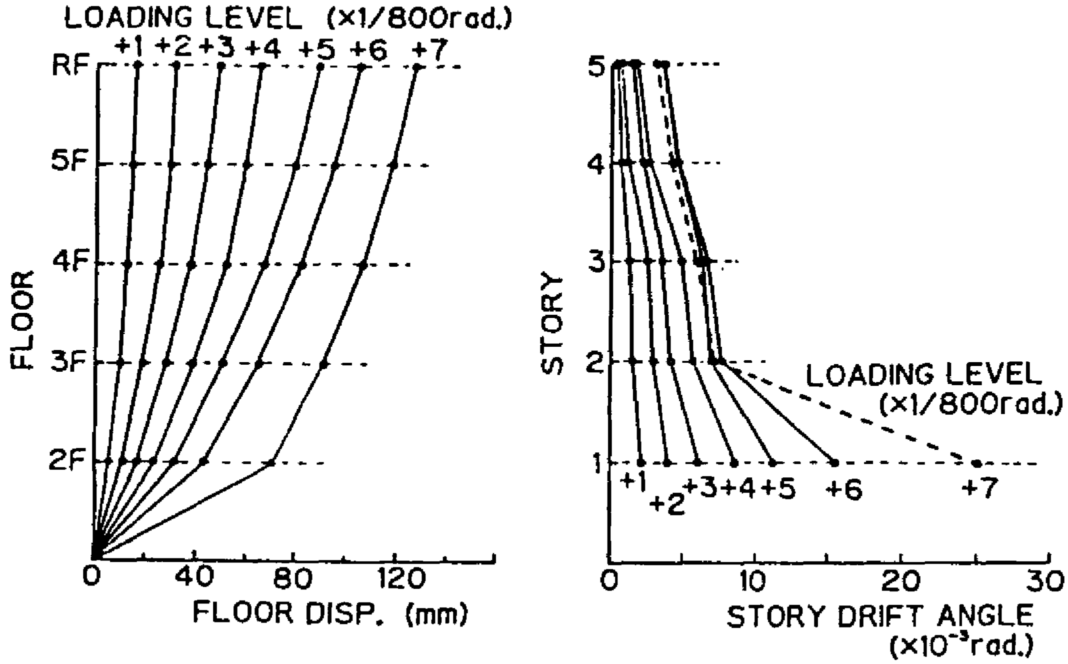


Fig.15 Peak Response to Taft EW 1952 (0.3g)



a) Building Deflection Mode

b) Loading Level vs. Story Drift Angle

Fig.16 Progressive Deformation History

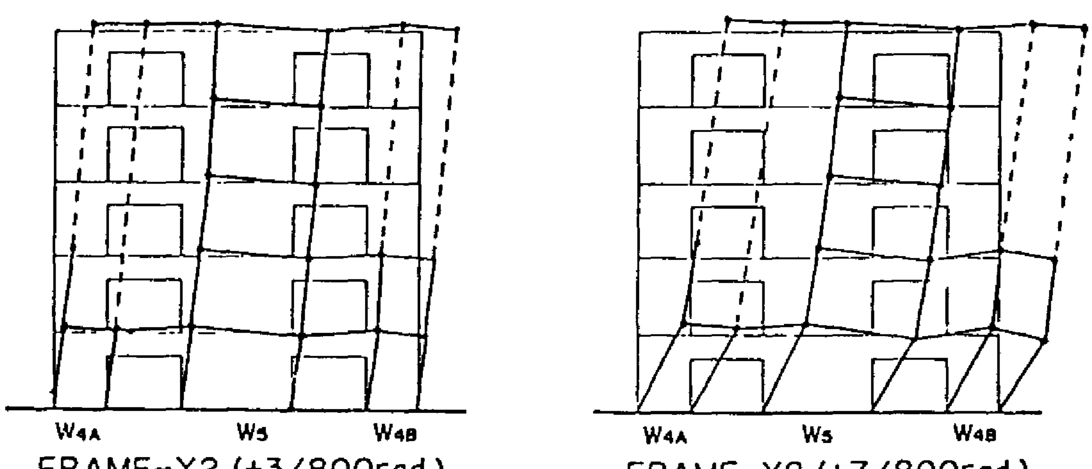
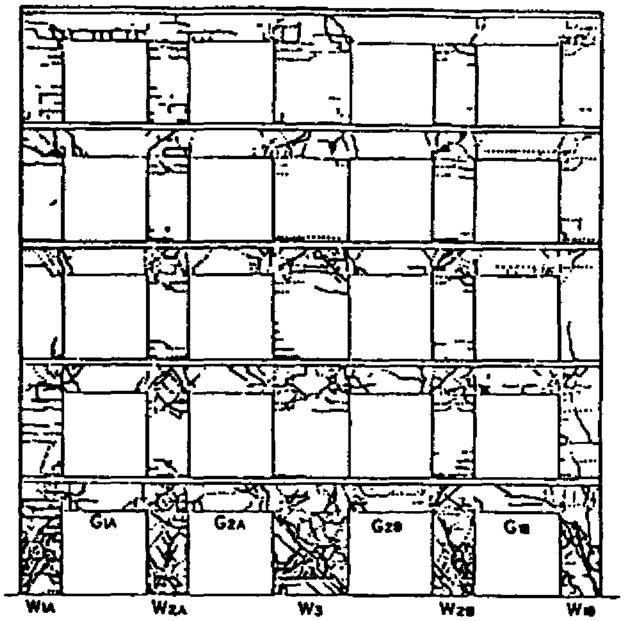
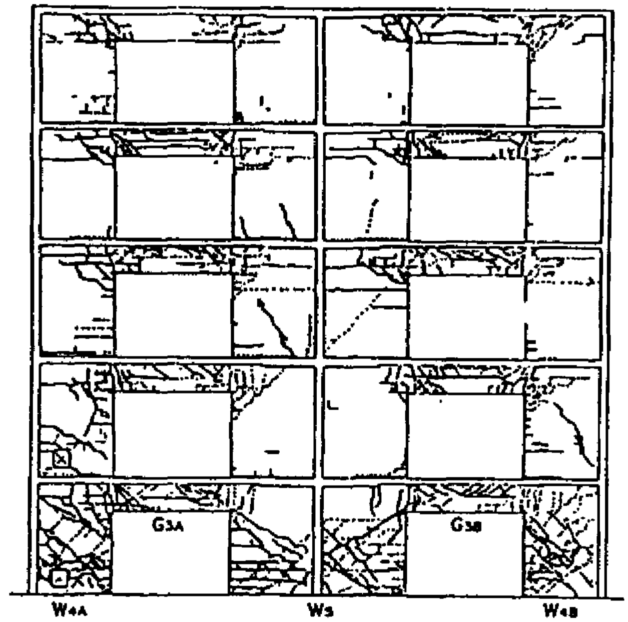


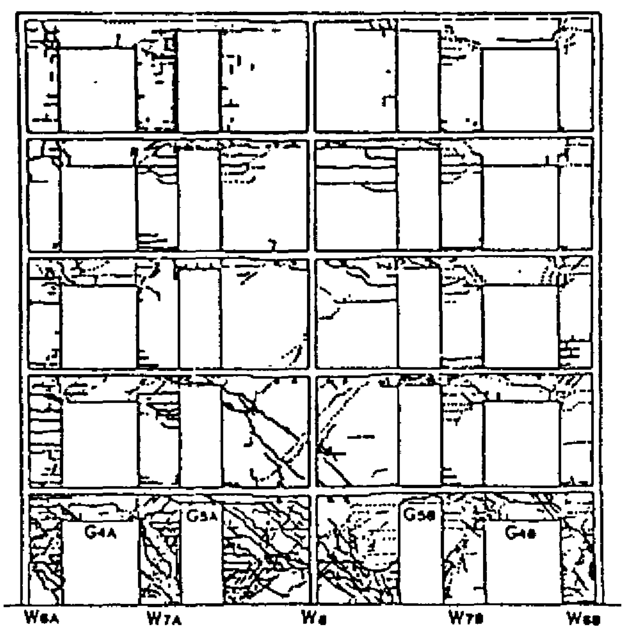
Fig.17 Overall Deflection Patterns of Frame Y2



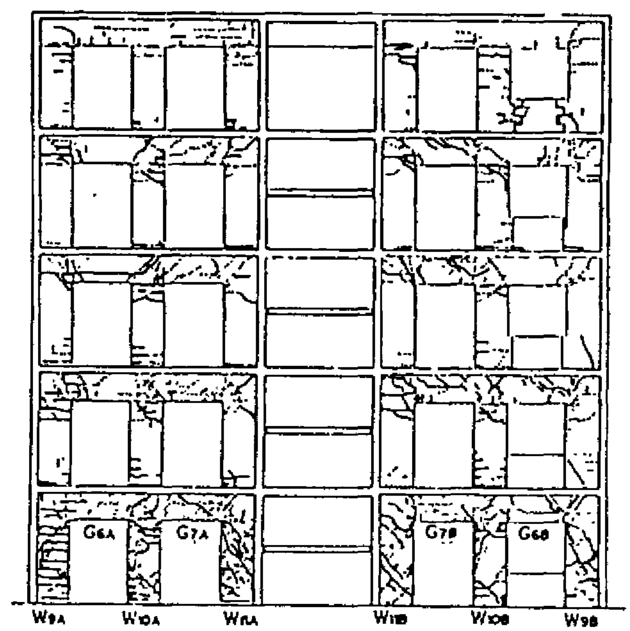
a) Frame Y1



b) Frame Y2



c) Frame Y3



d) Frame Y4

Fig.18 Ultimate Crack Patterns in Lateral Load Bearing Frames

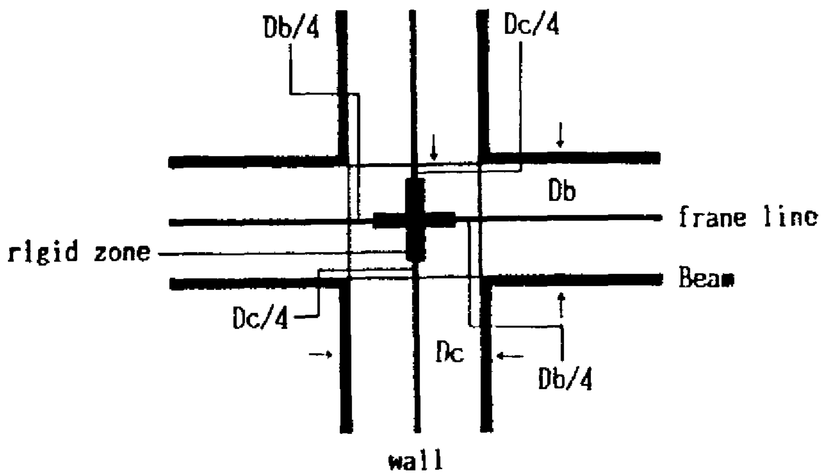


Fig.19 Assumption of rigid zone

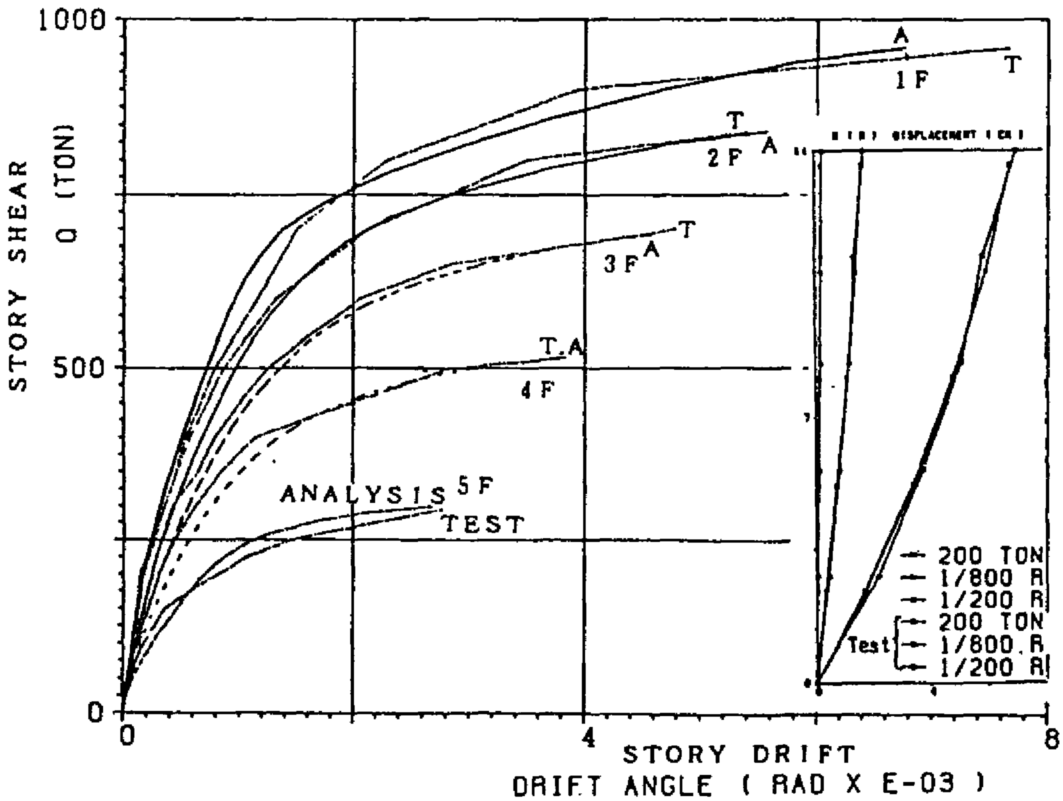


Fig.20 Story Shear vs. Story Drift Relationship