

Introduction to U.S. - Japan Cooperative Earthquake Engineering Program

By J. Penzien, H. Umemura, M. Watabe, and R. Hanson

Synopsis: This paper presents the background of activities leading to the current U.S.-Japan Cooperative Earthquake Engineering Research Program on the seismic performance of building structures which is being conducted under the auspices of the U.S.-Japan Panel on Wind and Seismic Effects, United States-Japan National Resources (U.J.N.R.) Program with major financial support provided by the U.S. National Science Foundation (NSF), the Japanese Ministry of Construction (MOC), and the Japanese Science and Technology Agency (STA). Also presented is a brief description of the current program and a statement on the coordination effort which is the responsibility of a Joint Technical Coordinating Committee (JTCC).

Keywords: earthquake resistant structures; reinforced concrete; research; structural analysis; structural design.

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BACKGROUND

Following the 1968 Tokachi-Okii earthquake in Japan, during which numerous reinforced concrete school buildings of modern design suffered heavy damage, it was apparent that every effort should be made to improve new designs through (1) learning as much as possible from the experience of the Tokachi-Okii earthquake, (2) reviewing and considering possible changes in building codes, (3) improving design and construction practices, and (4) initiating programs of needed research. Responding to this need, a joint seminar under the sponsorship of the U.S.-Japan Cooperative Science Program was held in Sendai, Japan, during the period 21-26 September 1970 for the purpose of (1) reviewing, in depth, the causes of damage sustained by modern school buildings during the Tokachi-Okii earthquake, (2) examining design and construction methods, and (3) identifying and defining needed programs of research which could be conducted more effectively on a cooperative basis. The official participants of this seminar were H. Aoyama, University of Tokyo, R. Hanson, University of Michigan, P. Jennings, California Institute of Technology, H. Kobayashi, Tokyo Institute of Technology, K. Lee, University of California, Los Angeles, K. Ogura, Meiji University, K. Ohno, Hokkaido University, J. Penzien, University of California, Berkeley, J. Roesset, Massachusetts Institute of Technology, T. Shiga, Tohoku University, M. Sozen, University of Illinois, H. Umemura, University of Tokyo, and M. Wakabayashi, Kyoto University. J. Penzien and H. Umemura served as coordinators of the seminar. The proceedings of this seminar

(Proceedings of the U.S.-Japan Seminar on Earthquake Engineering with emphasis on the Safety of School Buildings - 466 pgs.) was published by the Japan Earthquake Engineering Promotion Society.

A second joint seminar under the sponsorship of the U.S.-Japan Cooperative Science Program was held in Berkeley, California, during the period 4-8 September 1973 for the purposes of reviewing (1) the causes of damages sustained by reinforced concrete structures during the 1971 San Fernando earthquake (2) current research on earthquake resistant design, (3) the safety of existing structures and means of upgrading their resistance, and (4) post-earthquake damage repair. The official participants of this seminar were H. Aoyama, University of Tokyo, V. Bertero, University of California, Berkeley, B. Bresler, University of California, Berkeley, G. Corley, Portland Cement Association, A. Cornell, Massachusetts Institute of Technology, R. Hanson, University of Michigan, M. Hirose, Building Research Institute, S. Ikeda, Tokyo Metropolitan University, P. Jennings, California Institute of Technology, J. Jirsa, University of Texas, K. Kubo, University of Tokyo, A. Mattock, University of Washington, J. Penzien, University of California, Berkeley, Y. Ozaka, Tohoku University, M. Sozen, University of Illinois, and M. Wakabayashi, Kyoto University. B. Bresler and K. Kubo served as coordinators of the seminar.

Prompted by informal discussions held at the Sendai seminar, a U.S.-Japan Cooperative Research Program on Earthquake Engineering with Emphasis on the Safety of School Buildings was established under the U.S.-Japan Cooperative Science Program for the period May 1973 - October 1975. The cooperative research in this program was conducted by B. Bresler, T. Okada, J. Penzien, and M. Murakami at the University of California, Berkeley, by A. Shibata and M. Sozen at the University of Illinois, and by R. Hanson and T. Nishikawa at the University of Michigan. J. Penzien and H. Umemura served as coordinators of this program.

A review meeting of the cooperative research program emphasizing the safety of school buildings was held at the East-West Center, University of Hawaii during the period 18-20 August 1975. The official participants of this meeting were H. Aoyama, University of Tokyo, B. Bresler, University of California, Berkeley, R. Hanson, University of Michigan, P. Jennings, California Institute of Technology, S. Kokusho, Tokyo Institute of Technology, T. Okada, University of Tokyo, J. Penzien, University of California, Berkeley, A. Shibata, Tohoku University, M. Sozen, University of Illinois, and H. Umemura, University of Tokyo. The nineteen technical papers presented at this meeting have been published by the Japan Earthquake Engineering Promotion Society (Proceedings of the Review Meeting, U.S.-Japan Cooperative Research Program in Earthquake Engineering with Emphasis on the Safety of School Buildings). J. Penzien and H. Umemura served as coordinators of the review meeting.

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Based on evidence provided by the presentation and discussion of the nineteen technical papers, it was quite apparent to all participants attending the Hawaii meeting that the cooperative research program had been highly successful. With this experience in mind, a set of RECOMMENDATIONS FOR U.S.-JAPAN COOPERATION IN THE FIELD OF EARTHQUAKE ENGINEERING was drafted and signed by all official participants of the meeting. Recommendations Nos. 2 and 3 read as follows:

"RECOMMENDATION NO. 2 -- Establish a Cooperative Research Program in Earthquake Engineering with Emphasis on Large-Scale Testing of Structural Systems under the sponsorship of the U.S.-Japan Panel on Wind and Seismic Effects, U.J.N.R. Program. This program would concentrate on controlled dynamic testing of full-scale buildings in the field and laboratory testing of large-scale building systems using the available specially designed testing floors and associated reaction walls and using a large-sized shaking table. These tests should be designed to provide needed information on force-deformation, energy absorption, and failure characteristics of such systems. Correlative and analytical studies should be made to improve mathematical modelling and computer-oriented dynamic analysis capability.

RECOMMENDATION NO. 3 -- Establish a Task Committee under the U.S.-Japan Panel on Wind and Seismic Effects, U.J.N.R. Program. The assignment given to this Task Committee should be to make detailed plans and recommendations for implementing Recommendation No. 2 which will ensure an effective research program of maximum benefit to both countries within the constraints of the sponsoring government agencies. In view of the time scale for constructing the presently planned test facilities and the major effort required to plan the program, the Task Committee should be appointed by the U.S.-Japan Panel on Wind and Seismic Effects at the earliest possible date. One American and one Japanese should be appointed as co-chairmen to head this Task Committee."

Parallel with the above described cooperative activities of university researchers, government delegates on the U.S.-Japan Panel of Wind and Seismic Effects, U.J.N.R. Program, were taking positive steps toward strengthening such cooperative efforts. At the conclusion of its sixth joint meeting held in Washington, D.C. during the period 15-17 May 1974, the panel adopted the following resolution:

"4. that increased effort should be made in the near future to encourage joint research programs, especially in the area of mutual utilization of research facilities and the exchange of researchers."

At the panel's seventh joint meeting held in Tokyo during the period 20-23 May 1975, justification of a cooperative large-scale testing program was stated in a paper "Earthquake Disaster Mitig-

gation: A Joint Research Approach" presented by C. C. Thiel and J. B. Scalzi as follows:

"2. Large Scale Destructive Testing of Structures

In the area of structural analysis and design we have relied heavily on our technical capabilities with theory and computers to develop concepts for design of structures to resist earthquake forces. Many of these concepts have evolved from post-inspections of earthquake damage and shake table results. As beneficial as these concepts are, there are many factors which cannot be evaluated by inspection or small scale tests.

Among the parameters for which better data is required are: three dimensional behavior of full size structures and individual components subjected to controlled seismic type forces, the determination of the structural damping characteristics caused by the various elements in a building, the connections of various structural components and equipment, the forces acting on various equipment caused by the interaction of the structures and the equipment, the attachment of non-structural items and other items which are either too large or too cumbersome to be handled on a shake table or confidently by analysis.

A plan to obtain the required data could be formulated by a Joint U.S.-Japan program consisting of: (a) large scale testing of existing structures of various types of materials and construction, (b) pseudo-dynamic tests on full size specimens or structures constructed to obtain specific data, (c) shake table verifications where required for the pseudo-dynamic tests or for specimens which may be considered full size.

At the present time a few projects are underway to investigate the behavior of masonry construction by pseudo-dynamic test methods. Large joint specimens of reinforced concrete frame and shear walls are being analyzed and tested by pseudo-dynamic test procedures.

The results of these tests could be verified by full size structural tests to evaluate the time behavior in a structural system. A program to extend the tests to full size structures to determine the parameters which cannot be evaluated otherwise would be most desirable and beneficial to both countries."

At its Tenth Joint Meeting, Washington, D.C., 23-26 May 1978, the U.S.-Japan Panel on Wind and Seismic Effects again indicated its strong support of the program by adopting the following resolutions:

"3. The Panel on Wind and Seismic Effects recognizes the importance of the U.S.-Japan Cooperative Program on Large Scale Testing and it urges early implementation of the program under the auspices of this Panel."

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In January 1976, J. Scalzi of the National Science Foundation encouraged J. Penzien to submit a specific proposal to NSF to implement RECOMMENDATION NO. 3 quoted above, i.e., to establish a Task Committee for the planning of a cooperative research program on large-scale testing. Prompted by this encouragement and the past recommendations for cooperative research, a proposal (UCB-Eng-4227) entitled "Planning a Cooperative Research Program in Earthquake Engineering with Emphasis on Large-Scale Testing of Structural Systems" was submitted to the Research Applied to National Needs (RANN) Directorate of NSF by the University of California, Berkeley. This proposal called for the planning effort to be conducted under the auspices of the U.S.-Japan Panel on Wind and Seismic Effects. NSF responded favorably to this proposal by issuing a Grant (No. ENV-76-80835) in the amount of \$85,700 for the period 15 July 1977 - 31 December 1979, with J. Penzien serving as Principal Faculty Investigator.

The planning group, consisting of H. Aoyama, University of Tokyo, V. Bertero, University of California, Berkeley, B. Bresler, University of California, Berkeley, G. Corley, Portland Cement Association, R. Hanson, University of Michigan, J. Jirsa, University of Texas, B. Kato, University of Tokyo, S. Kokusho, Tokyo Institute of Technology, E. Leyendecker, National Bureau of Standards, L. W. Lu, Lehigh University, S. Mahin, University of California, Berkeley, A. Mattock, University of Washington, Y. Ohsaki, University of Tokyo, K. Ohtani, National Center for Disaster Prevention, T. Okada, University of Tokyo, T. Okubo, Public Works Research Institute, J. Penzien, University of California, Berkeley, A. Shibata, University of Tohoku, M. Sozen, University of Illinois, Urbana-Champaign, H. Umemura, University of Tokyo, M. Wakabayashi, Kyoto University, and M. Watabe, Building Research Institute, held four meetings; Tokyo, Japan, 5-10 September 1977, San Francisco, California, 15-19 May, 1978, Tokyo, Japan, 18-23 December, 1978, and Berkeley, California, 9-14 July, 1979. Numerous individuals from universities, private practice, and government attended these meetings and made significant contributions to the planning program.

PLANNING GROUP RECOMMENDATIONS

The above planning group transmitted recommendations to the potential government sponsoring agencies (NSF, MOC, and STA) through its final report entitled "Recommendations For A U.S.-Japan Cooperative Research Program Utilizing Large-Scale Testing Facilities", Earthquake Engineering Research Center Report No. UCB/EERC-79/26, University of California, Berkeley, September, 1979, as follows:

"(1) It is recommended that a full-scale seven-story reinforced concrete building structure (including nonstructural elements such as curtain walls, partition walls, etc.) representing good current practice be tested in the Large-Size Structures Laboratory, Building Research Institute (BRI), Tsukuba New Town for Research and Education, Japan. The tests

should be conducted using a procedure intended to simulate dynamic response to prescribed seismic excitations. A computer-actuator on-line (pseudo-dynamic) test procedure should be considered for this purpose. After an initial series of tests limiting the response to light damage levels, the structure should be repaired and then retested under severe simulated seismic conditions to near collapse conditions. The objectives of this final test would be to assess the effectiveness of repair procedures and to determine the structures large deformation and failure characteristics.

Further, it is recommended that a series of coordinated experiments associated with the full-scale tests be conducted in Japan and the United States on reinforced concrete joint assemblies, walls, and frames, that one-tenth scale models of the full-scale seven-story test structure be tested under simulated seismic conditions using a single horizontal component shaking table, that a one-third scale model (including nonstructural elements) of the same full-scale test structure be tested under simulated seismic conditions using a two-component (vertical and one horizontal) shaking table, and that a full-scale three-story model be tested under simulated seismic conditions using a single horizontal component shaking table.

The results of all associated tests in both countries should be fully correlated with each other and with the results of the tests made in Japan on the full-scale seven-story structure. The objectives of these correlation studies would be to identify the relative merits of component tests, small-scale tests, and medium-scale tests in predicting prototype performance under medium to severe seismic conditions and to improve structural details leading to improved seismic resistant design.

(2) In recognition of the need for test results on prototype steel building structures, it is recommended that a full-scale seven-story building designed by the latest U.S. and Japanese codes be tested in the Large-Size Laboratory of the Japanese Building Research Institute in Tsukuba New Town. The test structure should be a representative office building with a perimeter moment-resisting frame and a braced interior core. It should be tested in several stages using a loading procedure simulating realistic seismic conditions. The pseudo-dynamic procedure should be considered for this application. At the final stage of testing, nonstructural elements such as curtain walls and partitions should be added to the structural system and the entire structure should be loaded to failure.

In support of the full-scale test, it is recommended that a series of complementary quasi-static tests be conducted in the U.S. and Japan on girder-to-column connections, column-to-footing connections, bracing members, composite floor

systems, planar composite beam-and-column assemblages, three dimensional beam-and-column sub-assemblages, and braced and unbraced frame bents, that one-tenth scale models of the prototype structure be tested under simulated earthquake ground motions on a shaking table in Japan, and that a one-third scale model of the same structure be tested on a large shaking table in the U.S. Results of the one-third scale model tests should be used to establish correlations between the full-scale test and the shaking table test methods.

The recommended associated tests in the two countries should be carefully planned so that they are fully complementary; thus, the results of these tests can greatly assist in predicting the response of the prototype test structure at all levels of load application. The results of the various quasi-static tests and shaking table tests performed in both countries should be correlated with each other and with the results of the prototype structure.

(3) It is recommended that experimental verification tests and corresponding analytical studies be performed in Japan and in the U.S. in a coordinated effort to develop the pseudo-dynamic test method to a level suitable for full-scale testing.

(4) It is recommended that a program of analytical studies be performed to assist in the design of the test specimens, to identify appropriate seismic excitations, and to correlate and evaluate the experimental results. These studies should be undertaken on a cooperative basis at various institutions in Japan and the United States. They should also be carefully coordinated with the experimental portions of the overall research program."

The overall objective of these recommendations is to improve seismic safety practices through studies to determine the relationship among full-scale tests, small-scale tests, component tests, and analytical studies.

IMPLEMENTATION OF PLANNING GROUP RECOMMENDATIONS

Phase 1: Reinforced Concrete Structures

The first phase of the above recommended program was implemented in 1980 with major financial support provided by NSF, MOC, and STA. The central focus in this phase was the testing of a full-scale seven-story reinforced concrete building structure in the BRI laboratory in Tsukuba. All tests on this structure were carried out by the BRI staff, with assistance from two U.S. researchers (D. Rea and J. Wight), under the direct supervision of M. Watabe. The total cost of this test program was approximately \$1.0 million. Approximately \$0.5 million was provided from the U.S. with the remainder provided in Japan.

Associated test programs as recommended were carried out in Japan and the U.S. Correlation studies using data from these programs, as well as data from the full-scale building test program in Tsukuba, are still being carried out at various institutions in both countries. Also, analytical and experimental work is still underway in both countries to develop the pseudo dynamic testing technique. The cost of the associated test and the pseudo dynamic testing development programs in the U.S. totals approximately \$2.2 million. The corresponding cost in Japan is estimated at \$ 1.5million.

The series of 12 papers following this introductory paper present significant results and findings of the major programs carried out under Phase 1. Since final project reports are still under preparation, these papers will serve as a progress report on the overall program.

Phase 2: Steel Structures

The second phase of the recommended program was implemented in 1982 with major financial support provided by NSF and MOC. The central focus of this phase is the current testing of a six-story (not seven as originally recommended) steel braced frame structure in the BRI laboratory. The tests on this structure are being carried out by the BRI staff, with assistance from two researchers (D. Foutch and C. Roeder), under the direct supervision of M. Watabe. The total cost of this program is estimated to be \$ 0.8million of which \$0.4 million will be provided from the U.S.

Associated test programs as recommended are now being carried out in both countries and the pseudo dynamic testing development work is continuing. The cost and effort going into these programs is roughly equivalent to the corresponding cost and effort of the reinforced concrete structures program under Phase 1.

COORDINATION OF PROGRAM

Coordination of the overall program is the responsibility of a Joint Technical Coordinating Committee (JTCC) whose members are appointed by NSF and MOC. The memberships of JTCC for Phase 1 - Reinforced Concrete Structures consisted of V. V. Bertero, W. G. Corley, H. J. Degenkolb, R. D. Hanson, P. C. Jennings, J. O. Jirsa, R. C. Johnston, L. W. Lu, S. A. Mahin, J. Penzien, D. Rea, R. L. Sharpe, M. A. Sozen, and J. K. Wight from the U.S. and H. Aoyama, T. Hisada, B. Kato, M. Kasajima, Y. Matsusima, T. Okada, M. Okamatsu, T. Okubo, M. Ozaki, S. Tani, H. Umemura, and M. Watabe from Japan. J. Penzien and H. Umemura served as co-chairman of JTCC and R. Hanson and M. Watabe served as Technical Coordinators.

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Static Tests on Shear Walls and Beam-Column Assemblies and Study on Correlation Between Shaking Table Tests and Pseudo-Dynamic Tests

By H. Hiraiishi, S. Nakata, Y. Kitagawa, and T. Kaminosono

Synopsis: This paper consists of three kinds of studies. One is static tests on three story shear wall assemblies to investigate the seismic behavior, particularly deformation performance and deformation mechanism, of flexural type shear walls. Second is static tests on flexural type beam-column assemblies designed by either U.S. design practice or Japanese design practice. Third is correlation study on shaking table tests and pseudo dynamic tests developed for full scale seven story R/C building test. Major conclusions derived from these studies are as follows; (i) Stretch of the boundary column under tension of the first story controlled the over all behavior of flexural type shear walls, (ii) The effect of slab reinforcements on bending capacity of beams was greatly remarkable, which caused pinching phenomena in a load vs. drift relation while top of the beam was under tension, and (iii) Satisfactory correlation between shaking table tests and pseudo dynamic tests was obtained after the restoring force characteristics with the effects of strain rate and stress relaxation was appropriately evaluated.

Keywords: beam-column frame; damping capacity; deformation; dynamic tests; earthquake resistant structures; reinforced concrete; shearwalls; static tests; stiffness.

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INTRODUCTION

Prior to full-scale seven story reinforced concrete building test, static tests of shear wall assemblies and beam-column assemblies were conducted. The correlation study between shaking table tests and psuedo-dynamic tests used in full scale seven story building test was also investigated.

In static tests, basic specimens were designed to represent lower three story shear wall portion and beam-column portions of full-scale seven story building. However the number of the specimens includes two for shear wall assembly and eight for beam-column assembly, because the major objectives of these studies were placed not only to evaluate basic behavior of seismic members of the full scale seven story building, but to investigate general behavior, particulary deformation performance and deformation mechanism, of flexural type shear walls and the effect of slab reinforcement and lateral reinforcement of the beam, on a load vs. drift relation of beam-column assemblies.

Four reinforced concrete specimens of two story and about half scale models, two each for shaking table tests and for pseudo-dynamic tests, were prepared to investigate the correlation between the shaking table tests and the pseudo-dynamic tests. For input excitation on the shaking table tests, the recorded accelerogram at the first floor of Tohoku University obtained during the Miyagi-ken earthquake of 1978 was applied. On the pseudo-dynamic tests, the input excitation for the on-line computer program was determined by the obtained accelerogram on the table during the shaking table test. Dynamic analyses were carried out, both with and without the effects of strain rate and stress relaxation.

PLANAR TESTS ON REINFORCED CONCRETE SHEAR WALL ASSEMBLIES

Outline of Test

Static tests of two reinforced concrete three story shear wall-connecting beam assemblies were carried out, in order to examine the seismic behavior of flexural type shear wall which is continuous from the first to seven story at the central part of the center section, in loading direction, of the full-scale seven story building. Loading was conducted by using three jacks so that similar stresses as those in full-scale seven story building might be produced. The major objective of this study is placed on deformation performance, and deformation mechanism of such a flexural type shear wall.

Test Specimens

Two test specimens of W1 and W2 are a half scale model of shear wall-beam assemblies of the lower three stories of the full-scale seven story building. A specimen consists of shear wall, connecting beams with half length, and slabs with a width as specified in the Building Code Requirements for Reinforced Concrete, Architectural Institute of Japan¹⁾. The reinforcement ratios of the specimen W1 corresponds to those of the full-scale seven-story building. The other specimen (W2) has the same crosssection and dimensions as W1, but increased reinforcement is provided to get higher strength.

Drawing of the reinforcement arrangements of W1 and a table of the reinforcement for W1 and W2 are shown in Fig. 1 and Table 1. The major difference in the reinforcement arrangements between W1 and W2 was the amount of longitudinal reinforcement in column, the amount in W2 being 2.5 times that of W1. As for the other reinforcements, i.e. the wall reinforcement, longitudinal and lateral reinforcement of the connecting beam, the arrangement of W2 was from 1.5 to 2.0 times those of W1.

The test results for the reinforcement and concrete are shown in Tables 2 and 3. The concrete was standard, using river sand and river gravel with a design strength of $F_c = 270 \text{ kg/cm}^2$. The values for the reinforcement are the mean values of three tests of the respective sizes. Stress-strain curves for the reinforcements are shown in Fig. 2.

Application of Force

The loading apparatus is shown in Fig. 3. The horizontal load was applied as a concentrated load at the top. The connecting beams were linked to each other by tie bars with both pinned ends to constrain their vertical displacement. Two center hole jacks were used to apply the vertical load on the left-hand side and right-hand side of the force transfer beam at the top of the third story. This system allowed the left-hand side and right-hand side vertical loads (N_L and N_R) to vary with the

horizontal load, according to the formulae (1)-(4) shown below so that:

i) the axial stress in the first story wall would correspond to the axial stress of the full-scale seven story shear wall assembly ($\sigma_0 = 264,000 / (20 \times 500) = 26.4 \text{ kg/cm}^2$), and

ii) the moment gradient at hinge mechanism would correspond to that occurring when an inverted triangularly distributed external force was applied to the full-scale seven story shear wall assembly considering the effect of the connecting and transverse beams, and

iii) the effect of overturning moment of the connecting beams would be taken into account. The beams was assumed to yield at flexural strength of a shear wall, and shear force in the beam was also assumed to be proportion to horizontal load of a shear wall.

Vertical loads of Test Specimen W1:

$$N_L = 24.6* - 0.356P(t) \quad (1)$$

$$N_R = 25.6* + 0.356P(t) \quad (2)$$

Vertical loads of Test Specimen W2:

$$N_L = 24.6* - 0.364P(t) \quad (3)$$

$$N_R = 25.6* + 0.364P(t) \quad (4)$$

(* The difference of 1 ton between N_L and N_R is due to the weight of the loading apparatus.)

The bending moment diagrams for the test specimens, obtained according to the foregoing loading method, are shown in Fig. 5. The bending moment diagrams at maximum flexural strength are shown in Fig. 6 together with a target bending moment diagram (see Fig. 4). Because of limitation in the loading apparatus, for specimen W2, the control of the vertical load was possible only up to 58 tons, which was just short of the yield load.

Crack Patterns and Modes of Failure

Fig. 7 shows the final crack patterns of W1 and W2. Flexural cracks were observed at the base of the column for specimen W1 in the first story at a horizontal load of $Q=12$ tons ($\bar{\tau}=Q/tl=4.8\text{kg/cm}^2$) and 14 tons ($\bar{\tau}=5.6\text{kg/cm}^2$) in both positive and negative loadings, respectively. W2 had a flexural crack at the base of column at a positive load of 10 tons ($\bar{\tau}=4\text{kg/cm}^2$) and at a negative load of 20 tons ($\bar{\tau}=8\text{kg/cm}^2$). Thereafter, flexural cracks corresponding to interval of the hoop spacing was observed gradually in the upper part of the column. In W1, a flexural crack produced at a position of above 1.5 times column depth from the base developed into a shear crack (which should be rather called as flexural shear crack) in the wall, under both positive and negative loads of 24 tons ($\bar{\tau}=9.6\text{kg/cm}^2$). In W2, a similar shear crack in the wall was produced under a positive load of 16 tons ($\bar{\tau}=6.4\text{kg/cm}^2$) and a negative load of 20 tons ($\bar{\tau}=8\text{kg/cm}^2$). Increased load was accompanied by shear cracking in the wall, while flexural cracks in the column developed upward. On the second and third stories, shear cracks in the wall were produced

almost simultaneously under a load of 32 through 36 tons ($\tau=12.8\sim 14.4\text{kg/cm}^2$). These shear cracks were of an angle of about 30° , smaller than the angle of 45° of cracks produced by pure shear, but greater than the angle of the shear cracks in the first story.

The foregoing was the phenomena observed before a tip drift angle of $1/400$. Up to this stage, all of the connecting beams had flexural cracks. At a tip drift angle of $1/100$, clear shear cracks were observed in the first story column of both W1 and W2, and crushing of the concrete was observed at the base of the column and at the compressive bottom sides of the connecting beams. In W1, the concrete of the first story column began to break up significantly due to shear compressing at a tip drift angle of $1/50$, and with three repetitions under a tip drift angle of $1/50$, fracture of the longitudinal wall reinforcement took place. Thereafter, at tip drift angle of $1/40$, the majority of the vertical wall reinforcement and one longitudinal reinforcement of the column fractured, and during unloading, the concrete at the base of the column failed in compression. In W2, on the second repetition under a tip drift angle of $1/50$, the concrete at the base of column failed in compression as in W1.

The test specimens had no beams in the wall so that shear cracks produced in the upper and lower stories penetrated through the slabs to form continuous cracks throughout the upper and lower stories. However, these cracks were of small width, and absence of beams in the wall were not considered to have had appreciable effect on the behavior of the shear wall.

Strains in the Reinforcements and Concrete

Fig. 8 shows the yielding patterns of the reinforcement in W1 and W2. The longitudinal reinforcements at the base of the column and most of the vertical wall reinforcement yielded before a tip drift angle of $1/400$ and entire vertical wall reinforcements at the base yielded by $1/200$. Almost all the longitudinal reinforcement of connecting beams, including the slab reinforcement, yielded before a tip drift angle of about $1/200$. The lateral reinforcement, at the base of the column yielded after a tip drift angle of $1/200$. Concerning horizontal wall reinforcement, it was observed that in W2, horizontal reinforcement on the first story began to yield after a tip drift angle of $1/200$ and, before a tip drift angle of $1/100$ was reached, almost all of the horizontal reinforcements on the first story had yielded, while in W1 only the horizontal reinforcement at the lower part of the wall yielded at a tip drift angle of $1/200$.

Load vs. Deformation Relationship of Shear Wall and Connecting Beams

Figs. 9 and 10 show the relation of horizontal load Q to tip drift of W1 and W2. Both W1 and W2 gave a stable spindle-shaped loop and had no notable decrease in the strength for load repetitions within a tip drift angle of less than $1/100$.

Further, W1 and W2 had substantially the same interstory drift angles at each story.

Fig. 11 shows the relation of top vertical displacements $L\delta_v$ of the column of the third and first stories to a horizontal load Q at the loading side for W1. At the top on the tensile side of the third story, stretch of column was 5cm or more for a tip drift angle of $1/50$. However, on the compression side, the longitudinal shortening was less than 1cm. The vertical displacement of the second floor is not much different to that of the top of the third story, so it may be said that the greater part of the vertical displacement occurred on the first story. Similar results were obtained for W2 until the tip drift angle of $1/100$. After a tip drift angle exceeded $1/100$, the shear deformation appeared significantly for W2. Fig. 12 shows the relationships on the skeleton curve during positive loading between a drift angle of the first story and chord rotation at the top of the first story. The chord rotation is nearly equal to a drift angle.

From the foregoing, it can be pointed out that deformation of W1 and W2 showed bending yielding type behavior where deformation was mainly governed by the rotation due to stretching of the first story column under tension, and the second and third stories displayed a rigid behavior. Such deformation mechanism of a shear wall is represented by a truss model shown in Fig. 13. The truss members of the second and third stories are assumed to be rigid. It is also assumed that shortening of the boundary column under compression and diagonal member, and stretching of the beam are negligible, and tensile chord member (boundary column under tension) is expressed as a non-prismatic elasto-plastic member.

Fig. 14 shows some examples of the relations of the shearing force vs. drift angle of the beam, of W1 and W2. The beams were subject to yield at a drift angle of $1/200$ and to great deformation thereafter. While tension existed on the bottom side of the beam, a drift angle approximately equal to that of the shear wall were produced, and while tension existed on the top side of the beam, a drift angle about two times that of the shear wall was observed. This characteristic presented a remarkable pinching phenomenon.

The test specimens included no transverse beams. However, considering the amount of stretching and shortening of the columns on the tensile and compressive sides, it seems sufficient for calculating the restraining force by the transverse beam at maximum flexural strength of the shear wall to use the shear force at yielding of the beam for the tensile column side and a small value of the shear force for the compression column side.

Strengths

The cracking strengths and ultimate strength of the test specimens are shown in Table.4. The analytical values in the

table are based on the following formulae.

i) Shearing force at flexural cracking

$$M_{BC} = 1.8 \sqrt{F_c} Z_e + N Z_e / A_e$$

$$Q_{BC} = M_{BC} / (M/Q)$$

) Shearing force at flexural shear cracking

$$Q_{BSC} = 0.265 \cdot t \cdot D \sqrt{F_c} + M_{BC} / (M/Q - 0.5D)$$

) Shearing force at shear cracking

$$Q_{SC} = K_{tB} \cdot t \cdot \ell \cdot c \sigma'_t \sqrt{1 + \sigma_0 / c \sigma'_t}$$

$$K_{tB} = \frac{1 + 1.05t/B}{1 + 1.77t/B}$$

$$c \sigma'_t = 2.2 \sqrt{F_c} - 0.00546 F_c^{1.456} P_w^{0.075}$$

) Shearing force at maximum flexural strength

$$M_{FM} = 0.9 a_t \sigma_y D + 0.4 a_w w_s \sigma_y D + 0.5 ND \{1 - N/(BDF_c)\}$$

$$Q_{FM} = M_{FM} / (M/Q)$$

) Shearing strength

$$Q_{Su} = \left\{ \frac{0.053 P_{te}^{0.23} (F_c + 180)}{M / (Qd) + 0.12} + 2.7 \sqrt{s \sigma_{wy} P_{we} + 0.1 \sigma_0} \right\} b_e j$$

where,

A_e = effective cross-sectional area (cm²)

Z_e = effective section modulus (cm³)

N = axial force (kg)

M/Q = moment to shear at base of a shear wall (cm)

D = length between outer faces of both columns (cm)

t = thickness of wall (cm)

ℓ = length between centers of both columns (cm)

B = width of column (cm)

P_w = wall reinforcement ratio

a_t = gross cross-sectional area of longitudinal reinforcement in a column (cm²)

σ_y = yielding stress of longitudinal reinforcement of column (kg/cm²)

a_w = gross cross-sectional area of wall vertical reinforcement (cm²)

$w_s \sigma_y$ = yielding stress of wall reinforcement (kg/cm²)

σ_0 = average axial stress (kg/cm²)

$P_{we} = P_w t / t_e$

P_{te} = ratio of tensile longitudinal reinforcement area to total cross-sectional area of a shear wall (%)

t_e = total cross-sectional area of a shear wall/D (cm)

According to the analysis, both W1 and W2 reach yield strength without producing shear cracks, however many apparent shear cracks appeared in all stories in the test. These cracks are to be called flexural shear cracks because the angle of the crack was considerably smaller than 45°. With respect to the maximum flexural strength, the values agree well with those given by assuming that a plane section remains plane.

Conclusions of Tests on Shear Wall Assemblies

Following conclusions have been reached:

- i) Both test specimens, W1 and W2, showed a stable spindle-shaped load-deflection hysteresis.
- ii) The respective stories produced approximately equal drift angles. The deformation of shear walls was governed largely by the stretching of the tension side column at the first story.

Such deformation mechanism is represented by a truss model which consists of rigid members and a non-prismatic elasto-plastic member expressing the boundary column under tension.

iii) Horizontal wall reinforcements at the first story began to yield at the drift angle of $1/200$, and the majority of these reinforcements of W2 yielded after a rotation angle of $1/100$, although specimens showed flexural type behavior. The connecting beams at tension side column yielded generally at a tip drift angle of $1/200$, and produced a greater drift angle than that of the shear wall.

iv) Absence of the beams had little effect on the rigidity and strength of the shear wall.

TESTS OF REINFORCED CONCRETE BEAM-COLUMN ASSEMBLIES

Outline of Tests

One-half scale reinforced concrete test specimens were tested to study the behavior of beam-column subassemblies designed under Japanese and U.S. construction practices. The specimens represented second-floor interior and exterior beam-column assemblies and top-floor exterior beam-column assemblies of a seven-story full-scale test structure.

The placement of beam and column longitudinal reinforcement was the same in all specimens. The amount and arrangement of lateral reinforcement was varied in specimens following the U.S. and Japanese design requirements¹⁾. More web reinforcement in beams and columns was required by the U.S. design code²⁾.

Within a deformation range (displacement angle less than $1/100$) predicted by a nonlinear earthquake response analysis of the prototype structure, the behavior of the two specimens designed by the two codes was quite similar. Specimens with slab behaved in a manner different from the corresponding specimen without slab.

Test Specimens

Three types of beam-column subassemblies were chosen as test specimens;

- i) interior beam-column assemblies (I-series),
- ii) exterior beam-column assemblies (E-series), and
- iii) exterior beam-top column assemblies (T-series).

The inflection points were assumed to be located at the mid-span of beams and at mid-height of columns. The specimen was taken as the portion bounded by the inflection points. The dimensions of each test specimen were one-half those of the full-scale structure. Slab reinforcement in the half-scale specimens was placed in a single layer.

Three specimens in each series were designed using (a) Japanese design practice with slab, (b) U.S. practice with slab, and (c) Japanese practice without slab. The slab width was chosen to be 75cm on the basis of the effective width of T-beams. The effective width requirements are comparable in the two countries. Dimensions and reinforcement for test specimens are summarized in Table 5 and examples of their details of Specimens I-1, E-2 and T-2 are shown in Fig. 15. The major difference in U.S. and Japanese design requirements are the amount and arrangement of lateral reinforcement, and the method of anchoring beam longitudinal reinforcement in the beam-column connection. The amount of beam lateral reinforcement in the specimens was determined by the minimum requirements in both countries because the expected shear forces in the members were small. It is a Japanese construction practice in the exterior beam-column connection to bend down the beam bottom longitudinal reinforcement and to anchor the reinforcement in the lower column because the concrete is normally cast from a slab face to the next slab face above at a time.

High-early strength concrete was used in the specimens. All specimens were cast outdoors, from the same batch of concrete, in the up-right position. The average compressive strength of concrete was 340kg/cm^2 , which was 20 percent higher than the specified strength of 270kg/cm^2 . The beam-column subassemblies were tested at 17 to 55 days from concrete casting. The increase in concrete strength was observed to be small during this period. Reinforcing bars (D6, D10 and D13) were the same ones as used in shear wall assembly specimens.

Loading

Varying forced displacements were applied at the free ends of the beams. The loading apparatus for series I (interior), E (exterior) and T (top) beam-column subassembly specimens are shown in Figs. 16 and 17. The top end of the series I and E columns and the free end of the series T beam was supported by a vertical roller. The other ends of all specimens were either supported through a mechanical hinge, or connected to a loading device through a mechanical hinge whose horizontal movements were restricted. The location of mechanical hinges and roller in the specimens corresponded to the assumed location of inflection points in the full-scale structure.

The level of axial stress was 38.4kg/cm^2 in series I specimens, and 36.5kg/cm^2 in series E specimens. No constant axial stress was applied in series T specimens, but the column

axial stress varied with lateral load. The axial load fluctuated during the experiment by $\pm 12\%$ from the specified level.

The loading program is schematically shown in Fig. 18. In a series I specimen, the displacements at both beam ends were moved by the same amplitude, but in the opposite direction.

Test Results

Fig. 19 shows cracking patterns of Specimens I-1, E-1 and T-2 after test. In both I-series and E-series test, many flexural crack in the beams and minor cracks in the columns were observed. Slab cracks developed perpendicular to the beam axis extending over the entire slab width. U.S. and Japanese design requirements for amount and arrangement of lateral reinforcement caused little difference in cracking pattern. In T-series test where the beam and column were subjected to changing axial force in proportion to their shear force because of loading system shown in Fig. 16, many flexural cracks appeared in column as well as in beam of both specimens. When the slab was subjected to tensile stress, the number of flexural cracks in the column was larger than that in the beam.

Figs. 20 through 23 show story shear vs. interstory displacement relationships of Specimens I-2, I-3, E-2 and E-3. The relationships of Specimens I-1 and E-1 whose lateral reinforcements were small compared to those of Specimens I-2 and E-2, respectively, were almost identical to those of Specimens I-2 and E-2, respectively, up to a displacement amplitude of $R=1/50$. However, a little strength decay was observed in Specimens I-1 and E-1 in the final loading cycle of a displacement amplitude of $R=1/30$ as shown by a dot and chain lines in Figs. 20 and 22. The displacements of column and beam-column connection of Specimens I-3, E-1, E-2 and E-3 remained in elastic region. A ratio of displacement of the beam to interstory displacement became bigger according to increase in interstory displacement, and accounted for 95-98% at $R=1/50$. A shear force higher than the theoretical shear strength of the column at flexural cracking was produced in the column of Specimens I-1 and I-2. Displacement of the column increased and the ratio of displacement of the beam to total interstory displacement was 90-92%.

The reinforcement strains were measured at critical sections of beams and columns. In Figs. 20 through 23, the symbols $\blacktriangle$ and $\triangle$ mean the stages where yielding of longitudinal reinforcement of beam and slab reinforcement was observed, respectively. And dashed lines show theoretical yielding capacity of specimens. Specimens E-1 and E-2 yielded around these lines, while Specimens I-2 and I-3 yielded at lower load. These yielding strengths were determined by yielding of beam whose bottom was in tension. Tensile yielding of beam top reinforcement and slab reinforcement, particularly that of Specimens I-1 and I-2 was observed later. The slab reinforcement started to yield at the central part, and all yielded at maximum displacement. The column longitudinal

reinforcement and anchoring beam longitudinal reinforcement in the beam-column connection remained elastic.

Table 6 shows theoretical and experimental ultimate strengths. Experimental ones correspond to vertical loads applied at both beam ends as shown in figure, and theoretical values were calculated according to formulae shown below the Table. In the theoretical results, the slab reinforcement were assumed to be fully effective for flexural strength of the beam when the top of the beam was in tension. Specimens of I and E series theoretically fail in flexure of the beam. All experimental strengths of specimens of I and E series were considerably higher than the theoretical ones. Particularly, experimental strength of P2 of Specimens I-1, I-2 and E-1, I-2 which had the slab was significantly higher than theoretical values. This result means that slab increases strength of beam even when the bottom of beam was in tension. Axial force of beam and column of specimens of T series increased in proportion to load, so flexural capacity of the specimens was obtained by solving simultaneous equations. In Table 6, the mark *3 shows that specimen fails in flexural of beam and the mark *4 shows that it fails in flexure of column. The value marked by *5 is the strength where a half of column longitudinal reinforcing bars was considered as tensile reinforcing bars. The anchoring method of beam longitudinal reinforcing bars of Specimen T-1 is different from that of Specimen T-2, which caused a fairly different failure pattern between Specimens T-1 and T-2. Namely, as expected theoretically, yielding of critical section of column decided the strength of Specimen T-2 whose beam bottom longitudinal reinforcing bars were anchored upward in beam-column connection according to U.S. design practice. Meanwhile, anchoring beam longitudinal reinforcement downward into column increased strength of column of Specimen T-1, so that column did not yield at critical section and the damage spreaded around the top of column and beam-column connection. The experimental strength of Specimen T-1 was lower than theoretical one when bottom of the beam was in tension.

Conclusions of Beam-Column Assemblies

i) Within the deformation range up to the story deformation angle of $1/100$, the larger amount of web reinforcement and the placement of sub-ties in a column required by the United States design code did not improve the performance of the specimen because the deformation was not large enough to cause wide shear cracks, not to cause the crushing of concrete in beams and columns, nor to cause the buckling of column reinforcement. Therefore, the minimum requirements suggested by the Japanese design code were judged to be sufficient. Slightly better performance was observed from the specimens designed under the United States design code in a displacement range where the story deformation angle exceeded $1/50$.

ii) The slab contributed significant to the resistance of the subassembly, especially when the slab was stressed in tension.

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The increased resistance caused diagonal shear cracks in the beam, and a wide flexural crack in the lower part of the beam along the column face, causing a slip-type hysteresis loop at a story deformation angle greater than $1/100$.

CORRELATION STUDY ON SHAKING TABLE TESTS AND PSEUDO-DYNAMIC TESTS BY R.C. MODELS

Test Models and Input Motion

Four reinforced concrete specimens of two-story half-scale models, two each for shaking table tests and for pseudo-dynamic tests, were prepared. These four specimens had identical material properties and dimensions. Fig. 24 shows the plan and section of a test specimen. As seen in Fig. 24, the dimensions are 3m x 1m in plan, 3.9m in total height. Concrete is a normal weight with a design strength of 240kg/cm^2 . High early strength cement and deformed bars, SD30, having a nominal yielding stress of 3.0kg/cm^2 were used. The calculated fundamental natural period of the test model was 0.18sec.

For input excitation for the shaking table test, the recorded N-S component at the first floor of Tohoku University obtained during the Miyagi-ken earthquake of June 12, 1978 was applied. According to similitude law, the time axis was scaled by one-half, and the amplitudes were doubled. Fig. 25 shows time traces of waveforms both recorded at Tohoku University and used for the command signal to the shaking table. For the pseudo-dynamic tests, the exciting ground motions for the online computer program was determined by the recorded accelerogram on the shaking table.

Outline of Experiments

The schematic flow of the testing procedure for the correlation study is shown in Fig. 26. The loading sequence for the correlation study is shown in Table 7. In this table, two series of pseudo dynamic loading tests (P1 and P2) correspond to the first series of shaking table tests (D1). View of Specimens No. 1 and No. 2 under the tests is shown in Photo 1.

The shaking table is an electro-hydraulic servo controlled type, which is driven in one horizontal direction with a maximum stroke of $\pm 10\text{cm}$. The operational frequency range is zero to 50Hz and the maximum load is 20 tons. In order to investigate the dynamic behavior of test model under earthquake motion, the following items were measured; the relative displacements, the absolute displacements with servo and differential type transducers, and the absolute accelerations with U-gage type accelerometers. The test data were recorded by data recorders and monitored by electro-magnetic oscillograph recorders.

The specimen was loaded by two actuators for pseudo-dynamic test, with a maximum load of $\pm 100\text{tons}$, a maximum stroke of $\pm 500\text{mm}$

and a maximum velocity of 2mm/sec. The actuators were fixed to the reaction wall and load was applied at the center of each floor level. For input excitation, acceleration recorded on the shaking table in D1 series was used. A step-by-step integration in equation of motion for the computer-actuator online system was calculated by the central difference method. The time interval for integration was 0.005sec. The damping coefficient was zero in P1 test and 0.03 in P2 test. Inductance type transducers were used to measure the horizontal displacements. These measured displacements were used for actuator control. The transducers were attached vertically to the steel reference-frame. A steel wire, extending horizontally from each floor level, passed around a pulley before connecting with the transducers. The locations of measurements for strain and etc. were same as in the dynamic test.

Experimental Results

i) Shaking table test... Fig. 27 shows the accelerograms at each floor level of the test model under earthquake motions DR10 and of DR20. Time histories of relative floor displacements are shown in Fig. 28. Fig. 29 shows the shear force vs. story deflection at the first story. The maximum story drift angles of the first story were 1/228 and 1/51, respectively. Fig. 36 shows the final crack patterns of test model. Referring to the value of Table 8, the dynamic behavior for the shaking table tests are as follows; The natural period of the specimen, which was determined from a steady state vibration, was 0.23 sec. prior to the DR10 test. After the DR20 test, it became 0.71 sec. The former is 1.3 times and the latter is 3.9 times larger than the calculated period. Damping coefficients of 3.9% and 6.0% were obtained from resonance curves during the two series of steady state vibration tests, respectively.

ii) Pseudo-dynamic test... Fig. 30 shows the load history at each floor of the P1 series ($h=0\%$) under the earthquake motions of DR10 and DR20. The recorded relative displacements are shown in Fig. 31. Fig. 32 shows shear force vs. story deflection at the first story. The maximum story drift angles at the first story were 1/202 and 1/32, respectively. Fig. 37 shows the final crack patterns of test model.

Similarly, Figs. 33, 34 and 35 show the histories of loading, relative displacements at each floor and shear force vs. story deflection at the first story, respectively, for the P2 series ($h=3\%$) under earthquake motions of DR10 and DR20. The maximum story drift angles at the first story were 1/250 and 1/32, respectively. Fig. 38 shows the final crack patterns of test model.

Referring to the values of Table 8, the correlations between the two pseudo-dynamic tests with damping coefficients of zero and 0.03 are as follows; a) The maximum displacement of the two tests are nearly equal to each other in the test DR10. In the

test DR20, the maximum displacements in the one direction, with $h=0.00$ or $h=0.03$, were nearly equal, but in the other direction, the displacement obtained from the test with a damping coefficient of 3% was larger than that from the test with no damping, b) The maximum first story shear forces were nearly equal with each other in these two pseudo-dynamic tests, c) From the spectral ratios of displacement between records measured at the base and the top of the test model, the fundamental periods were 0.36 sec. in DR10 and 0.46 -0.47 sec. in DR20. The former is 2 times and the latter is 2.6 times larger than the calculated period, and d) Differences in response on the computer-actuator online system, due to the damping coefficients of zero or 3%, were small. The relative displacements at each floor obtained from the P1 and P2 series agreed fairly well with each other.

Comparison among Experimental Results

Comparing the values in Table 8, the results are as follows;

i) The maximum displacement and peak-to-peak displacement of the three tests (D1 , P1 and P2 series) were nearly equal in case of the DR10 test, ii) In the DR20 test, the maximum relative displacement obtained from two pseudo-dynamic tests (P1 and P2 series) were 1.4 to 1.7 times as much as that obtained in the shaking table test (D1 series). The maximum peak-to-peak displacements were 1.2 to 1.4 times larger than those from the shaking table test, iii) In the DR10 and DR 20 tests, the maximum first story shear force in the D1 series were generally larger than those in the P1 and P2 series. The ratio was 1.2 to 1.5 in the DR10 test and was 0.95 to 1.1 in the DR20 test, iv) In the DR10 test, the fundamental period in D1 series was smaller than those in the P1 and P2 series. In both tests, the difference was about ten percent, but in the DR20 test, the differences were small, and v) The final crack patterns in cases of D1, P1 and P2 series were similar with one another.

Fig. 39 shows the variation of average stiffness estimated during each cycle of shear force vs. story deflection in the DR10 and DR20 tests. In the DR10 test, the initial stiffness of the D1 series at the first story was larger than those in the P1 and P2 series, the ratio was 1.2 to 1.8. The initial stiffness at the second story was nearly equal to each other. In the DR20 test, the differences of the initial stiffness at the first story were lower than in the case of DR10.

Comparison between Analytical and Experimental Results

The dynamic response analyses in both the elastic and inelastic ranges were done by use of a lumped mass model with and without the effects of strain rate and stress relaxation. The modified D-Tri-linear model in which stiffness and strength are determined from the measured test data and the Maxwell Visco-elastic model was applied for the hysteresis rule. Figure 40 shows the phenomenon of stress relaxation in a specimen under pseudo-dynamic testing. Figs. 41 (a) -(b) show results for the

DR20 series, obtained from the experimental data and from the analytical model considering strain rate and stress relaxation effects. The results are as follows; i) In both DR10 and DR20 series, the response wave and the shear force vs. story deflection obtained from the analytical model in P1 and P2 series are in good agreement with the results obtained from the pseudo-dynamic tests, but do not agree with the results of the D1 and D2 series, ii) When stiffness and strength of the model were taken from the results obtained from the D1 and D2 series, the predicted response is in good agreement with that of the D1 and D2 series, the response wave and shear force vs. story deflection obtained from analyses considering the effects of strain rate and stress relaxation are in good agreement with the corresponding experimental results (compare Fig. 41 (a) with (c) and (b) with (d)).

Conclusions of Correlation Study

The results obtained from the two different experimental procedures and analytical results were compared. The correlations among these results were investigated. The results are summarized as follows;

- i) In the reinforced concrete test model, the influence on response of hysteresis damping is greater than that of viscous damping.
- ii) The influence of stiffness and strength on response is fairly critical.
- iii) Comparison of the test with the analytical predictions indicates that a good agreement between experimental and analytical results can be observed if the measured stiffness and strength of test specimens is used and consideration is given to the effects of strain rate and stress relaxation.
- iv) As a result, it is pointed out that the restoring force characteristics with the effects mentioned above should be evaluated accurately.

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Table 1--Reinforcement of Specimens W1 and W2

Specimen	Column				Wall
	b x D (cm ²)	a _u (cm ²)	a _b (cm ²)	p _s (%)	f ¹ x t (cm ²)
W1	25 x 25	2.7	7.93	0.43	250 x 10
W2	25 x 25	6.35	20.32	0.64	250 x 10

Specimen	Wall	Slab	Connection Beam		
	p _s (%)	a _s (cm ²)	a _{tb} (cm ²) ²	a _{tb} (cm ²) ³	p _s (%)
W1	0.356	2.56	2.14	1.43	0.43
W2	0.533	2.56	3.81	2.56	0.64

Note 1: Span of wall
 2: Cross sectional area of upper reinforcing bars
 3: Cross sectional area of bottom reinforcing bars

Table 2--Mechanical Behavior of Reinforcing Bar

	σ_y (t/cm ²)	σ_u (t/cm ²)	ϵ_y (%)	ϵ_{sc} (%)	ϵ_u (%)	E_s (10 ⁶ kg/cm ²)
D6	3.77	4.95	0.22	—	13.78	1.74
D10	3.75	5.32	0.195	2.4	14.39	1.89
D13	4.07	5.81	0.215	1.85	17.50	1.89
D22	3.81	5.96	0.19	1.45	17.95	2.02

σ_y = yielding strength

ϵ_y = yielding strain

σ_u = ultimate tensile strength

ϵ_u = strain at fracture

ϵ_{sc} = strain at start of strain hardening

E_s = Young's modulus

Table 3--Mechanical Behavior of Concrete

	Slump (cm)	F_c (kg/cm ²)	F_{sp} (kg/cm ²)	E_c (kg/cm ²)	ϵ_{cB} (%)
1st story	20.1	319.7	30.4	2.34×10^5	0.2080
2nd story	13.5	271.7	30.9	2.66 10	0.1462
3rd story	16.1	292.5	29.7	2.34 10	0.2022

F_c = compressive strength

F_{sp} = splitting tensile strength

E_c = Young's modulus

ϵ_{cB} = strain at maximum strength

Table 4--Strength

	Specimen	W 1			W 2		
	Floor	1st	2nd	3rd	1st	2nd	3rd
Shear force at first flexure cracking (ton)	Ex(P)	12	28	38	20	32	48
	Ex(N)	-14	-32	—	-20	-32	-48
	Theory	19.27	24.24	34.75	20.94	26.24	37.54
	Ex(P) / Theory	0.623	1.155	1.094	0.955	1.220	1.279
	Ex(N) / Theory	0.726	1.320	—	0.955	1.220	1.279
Shear force at first flexure shear cracking (ton)	Ex(P)	24	34	36	16	32	36
	Ex(N)	-24	-32	-32	-20	-32	-36
	Theory	38.32	47.69	76.09	40.46	50.50	80.64
	Ex(P) / Theory	0.626	0.713	0.473	0.395	0.634	0.446
	Ex(N) / Theory	0.626	0.671	0.421	0.494	0.634	0.446
Shear force at first shear cracking (ton)	Ex	—	—	—	—	—	—
	Theory	67.40	66.46	66.15	64.55	64.36	63.83
Flexure strength (ton)	Ex(P)	37.2	—	—	67.3	—	—
	Ex(N)	-42.7	—	—	-72.5	—	—
	Theory	30.55	—	—	54.55	—	—
	Theory*	38.6	—	—	68.6	—	—
	Ex(P) / Theory	1.218	—	—	1.234	—	—
	Ex(P) / Theory*	0.964	—	—	0.981	—	—
	Ex(N) / Theory	1.398	—	—	1.329	—	—
	Ex(N) / Theory*	1.106	—	—	1.057	—	—
Shear strength (ton)	Ex	—	—	—	—	—	—
	Theory	68.53	74.30	90.28	81.96	89.01	108.55

Ex = experimental result

Ex(P) = experimental result at positive loading

Ex(N) = experimental result at negative loading

Theory = analytical result given by i) - v)

Theory" = analytical result given by e-function method

Spec.	Slab	Arrangement of Reinforc. bars	Column Sp. Beam p_w
F-1	0	JAPAN CODE	0.290 0.228
F-2	0	U.S.A. CODE	1.310 0.130
F-3	X	JAPAN CODE	0.290 0.235
E-1	0	JAPAN CODE	0.290 0.228
E-2	0	U.S.A. CODE	1.310 0.130
E-3	X	JAPAN CODE	0.290 0.235
T-1	0	JAPAN CODE	0.290 0.228
T-2	0	U.S.A. CODE	1.310

Column; Design: main reinfrc. 3-D10 (T)
($p_t = 0.551$)
2-D10 (S)
($p_t = 0.431$)
Column 1-D10, 1-D13
($p_g = 1.271$, $p_t = 0.441$)

SPECIMEN	MEASURED		CALCULATED	
	P ₁ ^{#1}	P ₂ ^{#1}	P ₁	P ₂
I-1	3.35 (1.08)	1.04 (1.50)	3.10 ^{#2}	0.93 ^{#2}
I-2	3.34 (1.08)	1.48 (1.59)		
I-3	1.71 (1.21)	1.25 (1.34)		
E-1	3.47 (1.12)	1.40 (1.50)	3.10 ^{#2}	
E-2	3.49 (1.13)	1.33 (1.43)		
E-3	1.68 (1.19)	1.16 (1.25)		
T-1	7.89	1.98 (0.85)	7.71 ^{#3}	2.32 ^{#3}
T-2	7.39	2.59 (1.12)	5.08 (7.59) ^{#3}	

```

*2 (BEAM)    My = 0.9atσy D      (at; reinforcement in beam and slab)
*3 (BEAM)    My = 0.8atσy D + 0.5ND(1-N/bDFc) , N = P1cosθ
*4 (COLUMN)  My = 0.8atσy D + 0.5ND(1-N/bDFc) , N = P1sinθ
*5 (COLUMN)  at → ag/2

```

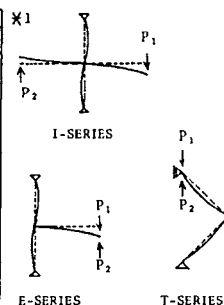


Table 7--Loading Sequence

Specimen	Run Number	Maximum Acc.	Note
NO. 1	DR 10	212 (gal)	
	DR 22	555	
NO. 2	DR 10	212	h = 0 (*)
	DR 20	555	
NO. 3	DR 10	212	h = 3 (*)
	DR 20	555	
NO. 4	DR 22	175	
	DR 21	80.7	

Remark : The former loading is intend to investigate elastic behavior of the specimen , and the latter , inelastic behavior.

Table 8--Experimental Results

		Displacement (mm)				Maximum Shear Force (ton)	Natural Period (sec)	Damping (%)
		Maximum		Peak-to-Peak				
		2F	RF	2F	RF			
DR10 212gal	STT	+8.2 -7.5	+12.3 -11.5	15.7	23.8	+4.7 -4.8	0.33	3.9
	POT-1 h=0%	+9.3 -8.1	+12.3 -11.8	17.4	24.1	+3.8 -3.5	0.36	2.9
	POT-2 h=3%	+7.5 -10.4	+9.7 -14.1	17.9	23.8	+3.1 -3.2	0.36	2.4
DR20 555gal	STT	+36.5 -29.0	+48.5 -50.1	65.5	98.6	+8.0 -7.0	0.46	6.0
	POT-1 h=0%	+58.4 -30.1	+82.7 -43.1	78.2	109.2	+7.6 -7.3	0.47	
	POT-2 h=3%	+58.3 -41.8	+81.1 -57.3	99.6	137.2	+7.8 -7.2	0.46	8.4

STT : Shaking Table Test POT : Pseudo - Dynamic test

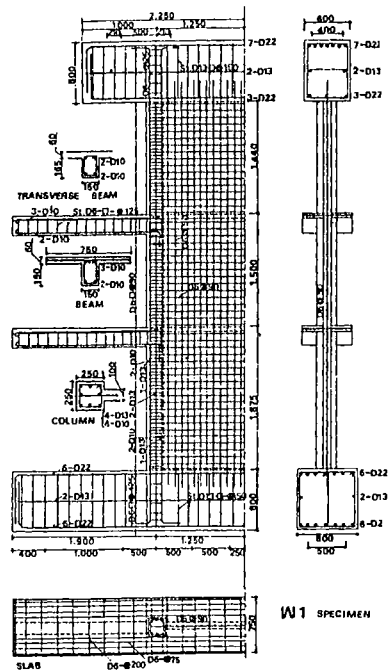


Fig. 1--Definitions and reinforcement of Specimen W1

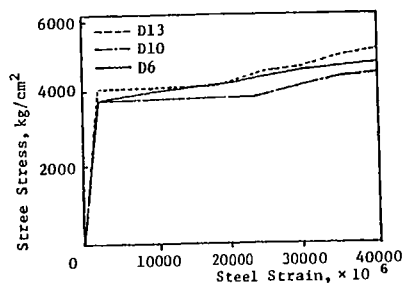


Fig. 2--Stress strain relationship of reinforcing bars

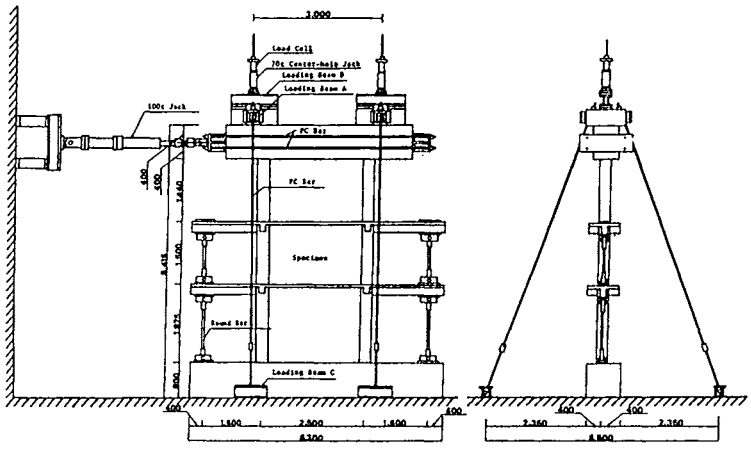


Fig. 3--Test set-up

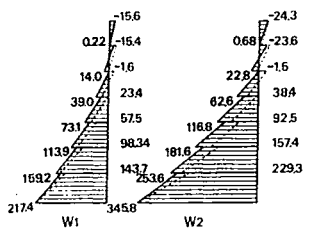


Fig. 4--Bending moment (t-m) at yielding of half scale seven story shear walls under inverted triangle lateral force distribution (dashed line shows the bending moment when the lateral force is assumed to be a uniform distribution)

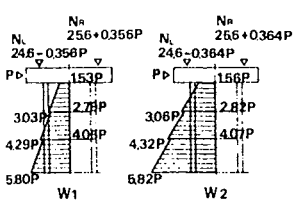


Fig. 5--Bending moment of the specimens

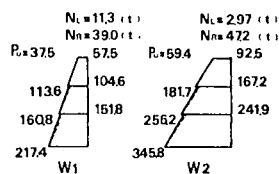


Fig. 6--Bending moment at yielding of the specimens (unit: t-m)
(dashed lines express target bending moment)

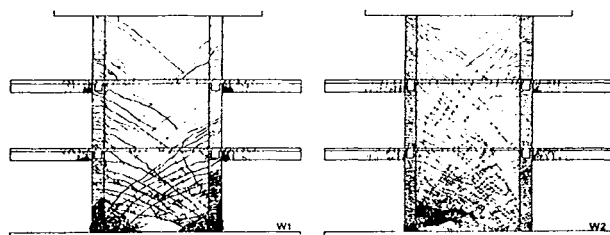
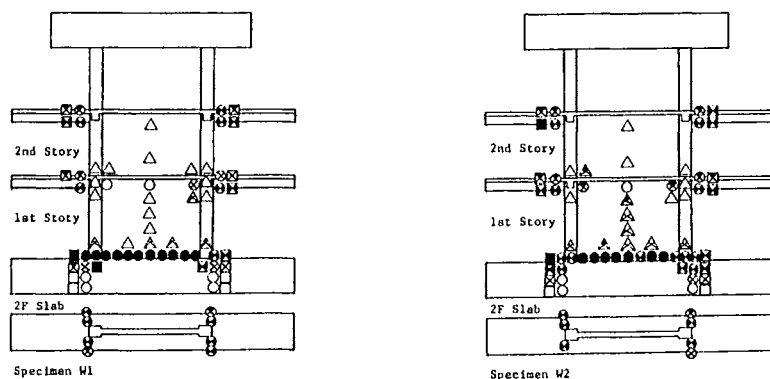


Fig. 7--Final crack patterns



Note: ● ○ = main reinforcement in the beam, slab and column, and vertical reinforcement in the wall, which are yielded at $R=1/400$, $1/400 \sim 1/200$, $1/200 \sim 1/100$, $1/100 \sim 1/50$ and not yielded, respectively.

■ □ = similar reinforcements to those above mentioned, but side is opposite.

▲ △ = hoop and horizontal reinforcement in the wall, which are yielded at $R \sim 1/400$, $1/400 \sim 1/200$, $1/200 \sim 1/100$, $1/100 \sim 1/50$ and not yielded, respectively.

Fig. 8--Yielding pattern of reinforcement of the specimen

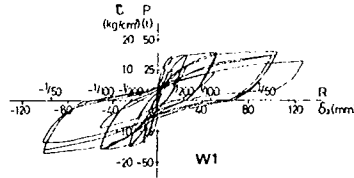


Fig. 9--Lateral load versus tip drift angle relationship of W1

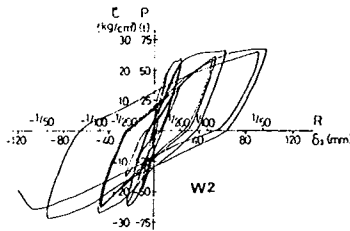


Fig. 10--Lateral load versus tip drift angle relationship of W2

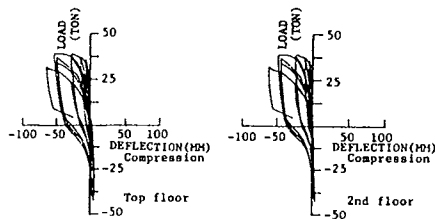


Fig. 11--Lateral load versus vertical displacement relationship of top and second floors at loading side of W1

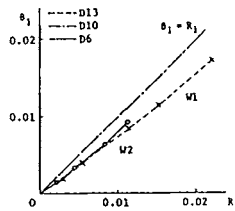


Fig. 12--Relationship between a drift angle (R_1) of the first story of the shear wall and a chord rotation (θ_1) at the top of the first story

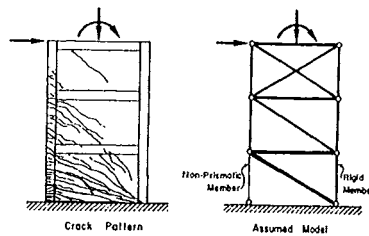


Fig. 13--Truss model

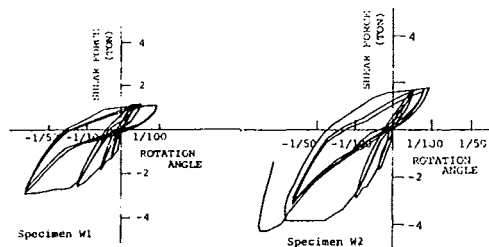


Fig. 14--Shear force versus drift angle relationship of connecting beam of second floor at loading side

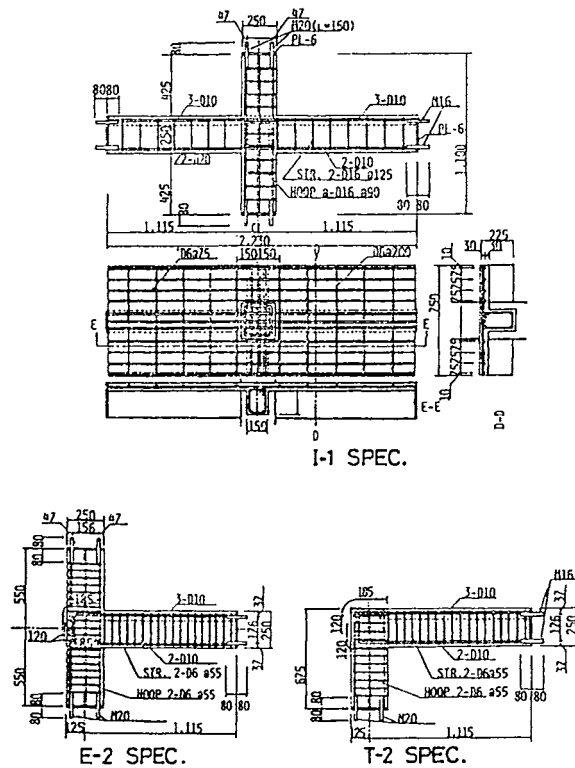


Fig. 15--Detail of specimens

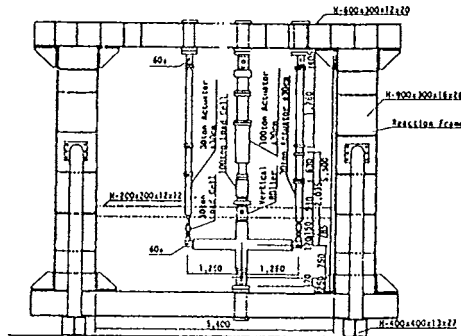


Fig. 16--Loading apparatus for I-Series Test

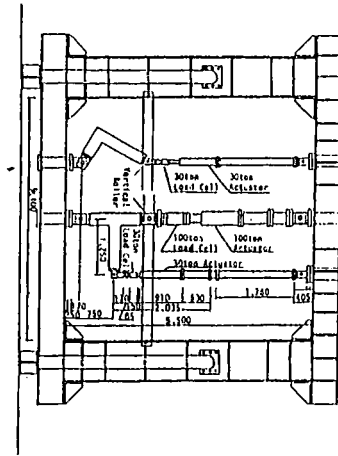


Fig. 17--Loading apparatus for E-and T-Series Test

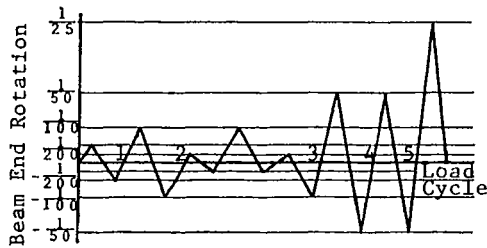


Fig. 18--Loading program

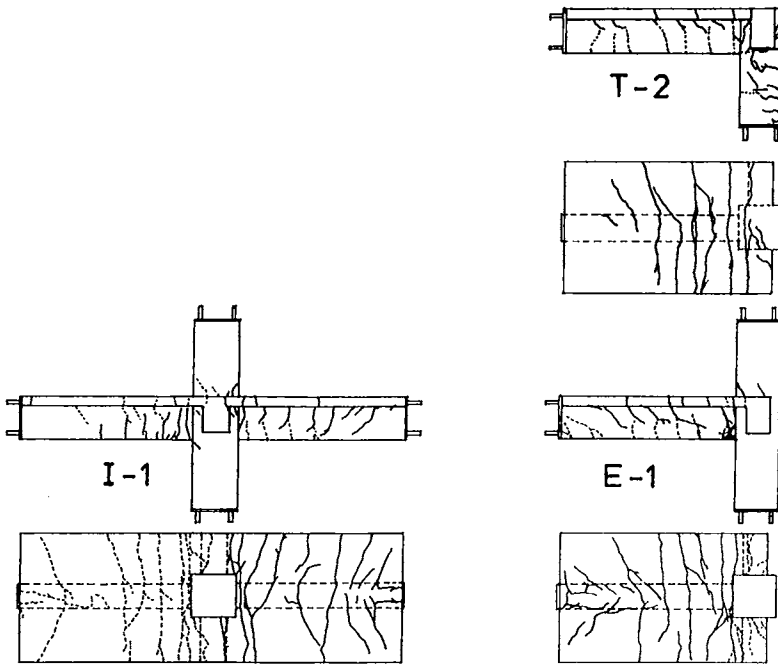


Fig. 19--Typical crack patterns

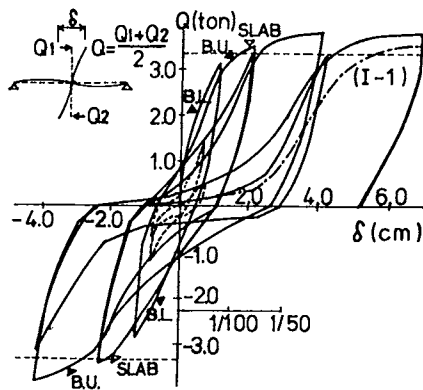


Fig. 20--Hysteresis Loop Specimen I-2

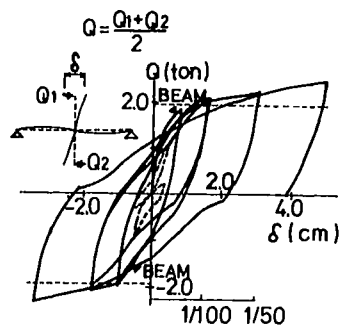


Fig. 21--Hysteresis Loop Specimen I-3

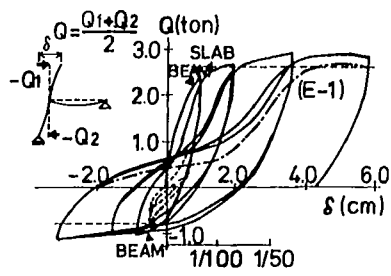


Fig. 22--Hysteresis Loop Specimen E-2

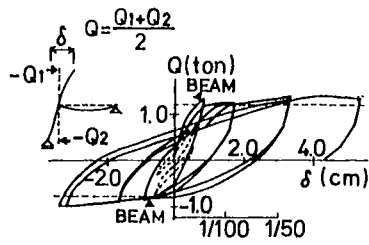


Fig. 23--Hysteresis Loop Specimen E-3

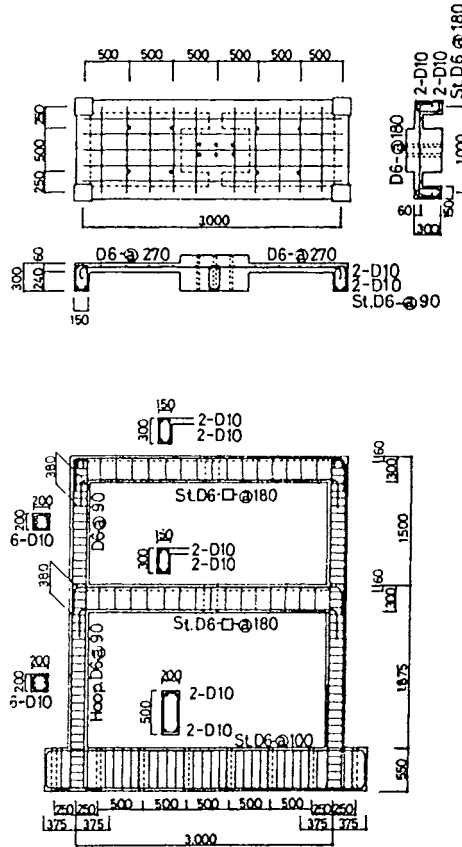


Fig. 24--Plan and section of test specimen

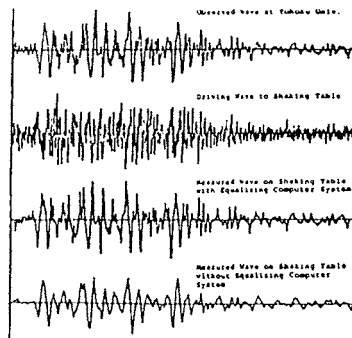
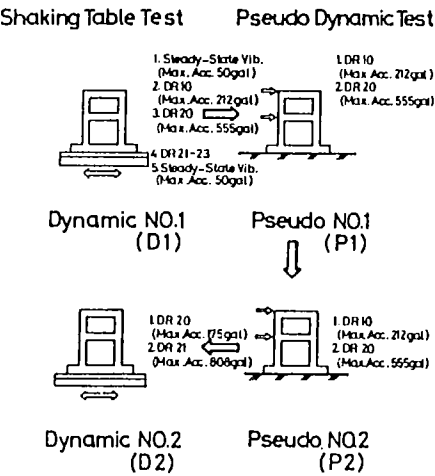


Fig. 25--Recorded and used accelerograms



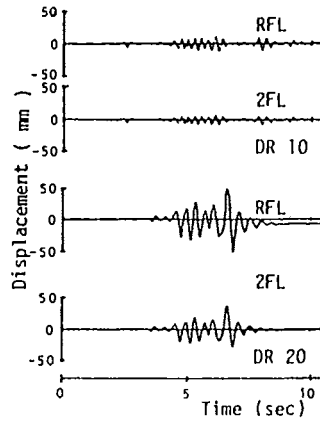


Fig. 28--Time histories of relative displacements

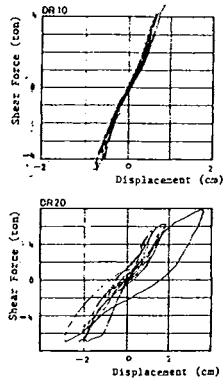


Fig. 29--Shear force versus story deflection

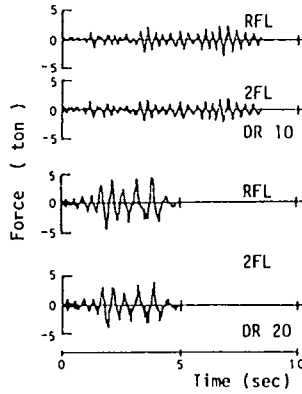


Fig. 30--History of loading

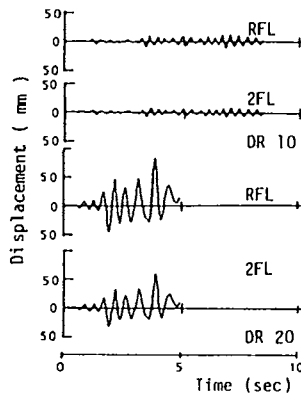


Fig. 31--Time histories of relative displacements

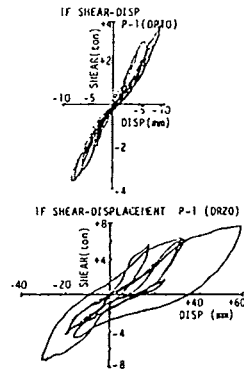


Fig. 32--Shear force versus story deflection

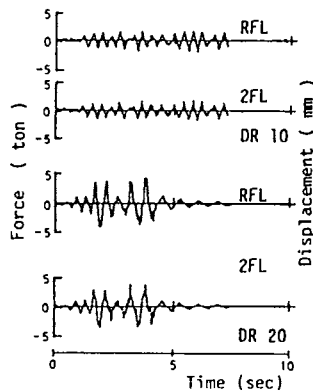


Fig. 33--History of loading

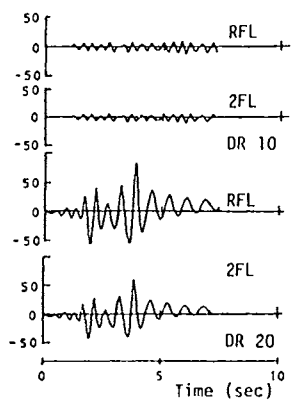


Fig. 34--Time histories of relative displacements

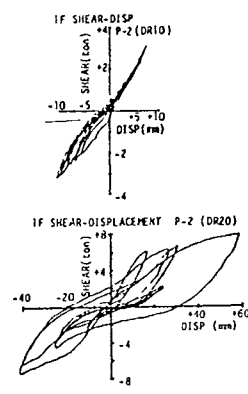


Fig. 35--Shear force versus story deflection

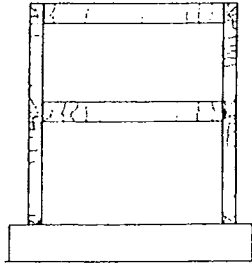


Fig. 36--Final crack patterns

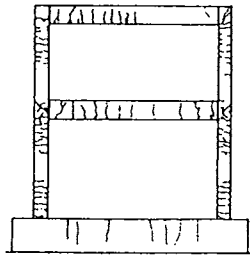


Fig. 37--Final crack patterns

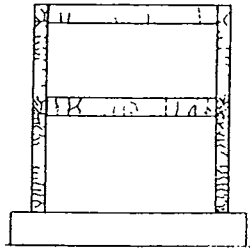


Fig. 38--Final crack patterns

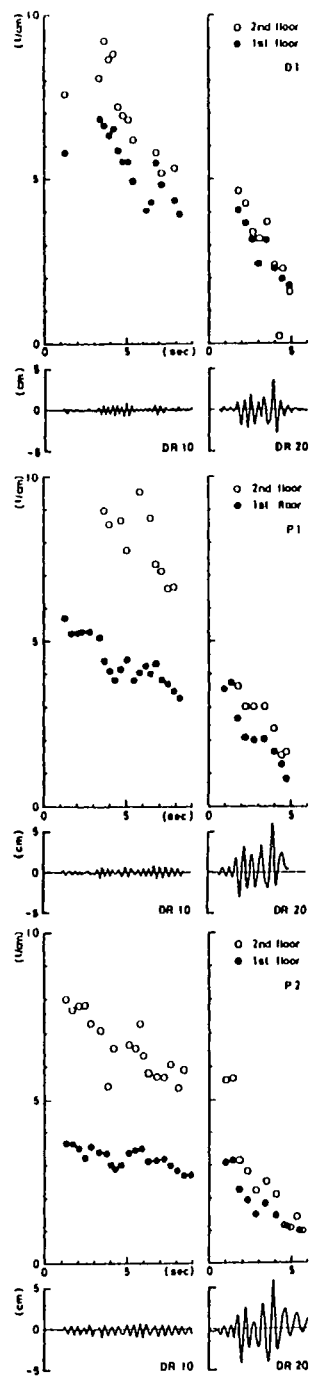


Fig. 39--Variation of average stiffness

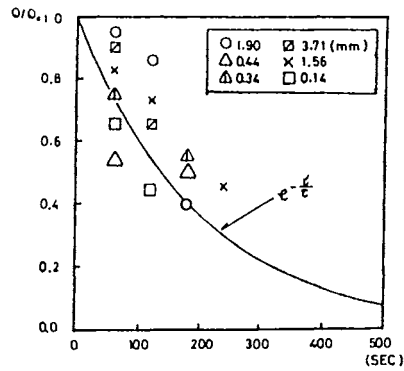


Fig. 40--Phenomenon of stress relaxation

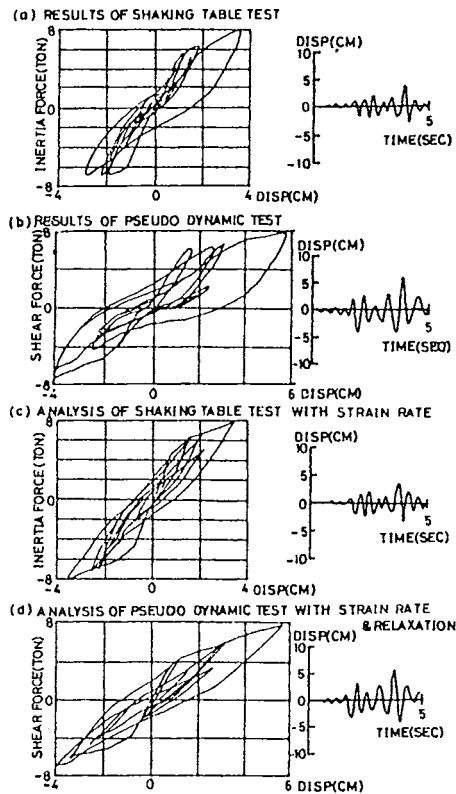


Fig. 41--Results of experiments and analyses

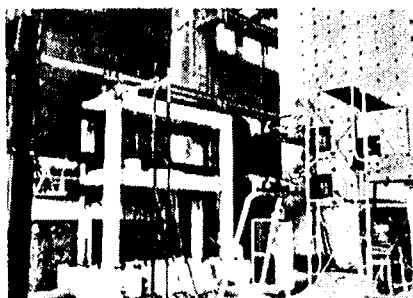


Photo 1--View of specimens

Construction and Instrumentation of the Full Scale Specimen

By J. Wight, S. Nakata, and T. Kaminosono

Synopsis: A summary of the construction and instrumentation of the full scale seven story reinforced concrete test structure is presented. The general construction procedure and differences between Japanese and U.S. construction practices are discussed. Cross section dimensions, reinforcement details and material properties for all structural members are presented. Instrumentation consisted of 200 displacement transducers, 415 strain gages mounted on reinforcing bars and 120 strain gages mounted on the concrete surface. The types and locations of the instruments attached to the test structures are defined and discussed.

Keywords: concrete construction; earthquake resistant structures; load tests (structural); measuring instruments; multistory buildings; reinforced concrete; strain gages; transducers.

ACI Fellow James K. Wight is an Associate Professor in the Department of Civil Engineering at The University of Michigan, Ann Arbor. He is Chairman of ASCE-ACI Committee 352 and Secretary of ACI Committee 307. His primary research interests are in earthquake resistant design of reinforced concrete structures.

Dr. Shinsuke Nakata is a member of the Structural Engineering Department of the Building Research Institute in Tsukuba, Japan. His primary research interests are in the behavior of reinforced concrete members and structures subjected to large cyclic load reversals.

Dr. Takashi Kaminosono is a member of the Production Department of the Building Research Institute in Tsukuba, Japan. His primary interests are in the application of the pseudo dynamic testing technique to large scale multidegree of freedom systems.

INTRODUCTION

The full size seven story reinforced concrete structure, which was tested as part of the U.S.-Japan Cooperative Earthquake Program (1), was constructed in the Large Size Structural Laboratory of the Building Research Institute, Ministry of Construction, Tsukuba, Japan. Construction started on September 17, 1980 and the last concrete was cast on January 12, 1981. A more detailed report on the building construction is given in reference (2).

The general layout of the building is shown in Figs. 1 through 3. Figure 1 is a general plan view and shows nominal span lengths. The location of the reaction wall is also shown in Fig. 1. The test structure consisted of three frames, A, B and C, parallel to the loading direction and four frames, 1, 2, 3 and 4, perpendicular to the loading direction.

A general elevation of frame B is given in Fig. 2. Spans 1-2 and 3-4 were open frames, but span 2-3 was a shear wall which had a nominal thickness of 200 mm (8 in.). The girders of spans 1-2 and 3-4 and the longitudinal reinforcement for those girders were not continued through the shear wall. Frames A and C were pure open frames and had dimensions identical to those given in Fig. 2.

Figure 2 also shows the floor level notation and story designations used in this report. Floor level Z0 is at the laboratory floor and the floor level numbers increase linearly up to level ZR at the roof.

A general elevation of frame 4 is given in Fig. 3. Both spans A-B and B-C had 150 mm (6 in.) thick shear walls, but the walls did not frame into the columns. A one meter (39 in.) gap was provided between the face of the columns and the edge of the