inter-story height. In other words, for the purpose of computing wall rotation, moment was assumed to distribute uniformly along the story height with an amplitude equal to the moment at the wall critical section. Cracking was to occur when the extreme tensile fiber strain became zero under the gravity load and overturning moment;

$$M_c = P\,(uL)\,/\,6$$

Yielding moment M_y was taken to be the full plastic moment; moment about the centroid of wall section caused by the yielding of all vertical wall reinforcement. The gravity load effect was included in computing the full plastic moment. The stiffness after yielding was taken to be 0.1 percent of the initial elastic stiffness.

Hysteresis Models

Axial-stiffness hysteresis model (Fig.10) was used for the two outside truss elements and central vertical spring element of the wall model.

A hysteresis model which dissipates small hysteresis energy was used for the rotational and horizontal springs at the base of the central vertical element of the wall model. The response point moved along a line connecting the origin and the previous maximum response point in each direction (Origin-Oriented Hysteresis Model; Fig.13). Once the response point reached the previous maximum point, the response point followed the skeleton force-deformation relation renewing the maximum response point. In this model, no residual deformation occurred, and the stiffness changed when the sign of resistance changed. No hysteresis energy was dissipated when the response point oscillated within a region between the positive and negative maximum response points. The skeleton curve of this model could be of any shape. A trilinear skeleton curve was used for the rotational and horizontal springs in the central wall.

MODELLING OF TRANSVERSE BEAMS

Member Model

The tensile boundary column of a wall elongated extensively with bending deformation, yielding a significant vertical displacement at a beam-to-wall joint node, whereas the vertical displacement of a beam-to-column joint node of an open frame was relatively small. Consequently, the transverse beam connecting the tensile boundary column and an adjacent parallel open frame was subjected to differential vertical displacement. The transverse beam, in turn, applied a downward vertical load to the boundary column of the shear wall, and upward vertical load to the column of the open frame. The effect of such transverse beams was modelled by vertical spring elements (Fig.14).

Skeleton Curve

The initial elastic stiffness of the vertical spring was evaluated as a fixed-fixed beam. Cracking and yielding forces were determined as a shear force acting in the transverse beam when both ends cracked or yielded simultaneously in flexure. Cracking moment, yielding moment and curvature of a T-shaped transverse beam section were evaluated in the same way as those of longitudinal beams. The effective width of 1,900 mm was determined referring to the results of the full-scale test (Ref.15). The stiffness after yielding was arbitrarily reduced to 3 percent of the initial stiffness. The numerical values of the stiffness properties of the vertical spring are listed in Table 8.

Hysteresis Model

The Takeda hysteresis model (Fig.9) was used for the transverse beam model.

METHOD OF ANALYSIS

The test structure was idealized as three parallel plane frames with the effect of the transverse beams connecting the shear wall boundary columns and adjacent parallel frames. Floor slab was assumed to be rigid in its own plane, causing horizontal displacements of all the joints in a floor level to be identical. The mass of the structure was assumed to be concentrated at each floor level. Vertical displacement and rotation were the two degrees of freedom at each joint. The frames and a shear wall were assumed to be fixed at the base. A routine stiffness method was used in the analysis.

A numerical procedure was developed to simulate the SPD test procedure. The mode shape, participation factor and the resistance distribution pattern were taken from the test as outlined in Ref.6. The mass of inertia for the response calculation was assumed the same as used in the test; i.e., from the second floor level to the roof level, 182.9, 169.7, 169.7, 169.7, 169.7, 169.7 and 152.9 ton. No damping was assumed in the test structure in the pseudo-dynamic response computation during the test, and the same was assumed in the analysis. The unbalanced forces, caused by overshooting at a break point of hysteresis curves, were released at the next time step.

The structure was observed to remain in an elastic range during test SPD-1 with the maximum roof-level displacement as small as 1/8630 of the total height. Therefore, the test run was not studied. Tests SPD-2 through SPD-4 were simulated continuously so that the structural damage in the preceeding test runs could be reflected in the analysis of the following test runs. Initial velocity in each test run was set to be null in a manner the same as in the test.

ANALYSIS OF TEST SPD-3

The analytical response of test SPD-3 was studied in details to examine the reliability of the analytical method because this test included a wide range of response including the yielding of various members and the shear wall. The roof-level displacement reached as large as 1/91 the total story height, a displacement which might be expected from this type of a structure during a "strong" earthquake motion. Studied were the analytical results of (a) reponse waveforms at the roof-level displacement and base shear, (b) hysteresis relation between base shear and roof-level displacement, (c) base shear-local deformation relations, and (d) force and deformation distribution at maximum response.

Response Waveforms

Observed and calculated waveforms of the roof-level displacement and base shear are compared in Fig.15. Response waveforms observed in test SPD-3 are shown in broken lines. Analytical response are in a good agreement with the observed response over the entire duration. Maximum displacement at roof-level was 238 mm from the test attained at 4.48 sec, while the calculated maximum amplitude was slightly larger (=248 mm) occuring at the same time. The period of oscillation elongated significantly after this time step both in the test and analysis. At 10.16 sec, the pseudo-dynamic free-vibration was started with existing residual displacement and null velocity. The period of the model appeared slightly longer than that of the structure.

Maximum base shear of 4,060 kN was attained at 4.48 sec in the test, whereas the computed value was 4,170 kN, slightly higher than the observed. Maximum base shear amplitude of a model can be easily controlled by choosing the yield resistance level and post-yielding stiffness. Parametric studies indicated that the combination of (a) the beam yield resistance computed with the slab reinforcement within 4300 mm width and (b) post-yielding stiffness equal to 3 percent of the initial elatic stiffness was most suited.

SDF Hysteresis Relation

As can be expected from a good correlation of the waveforms, the observed and computed hysteresis relations (Fig.16) are in a fair agreement, especially at the peaks of hysteresis loops. General shapes of the two curves are slightly different; i.e., the stiffness of the structure changed gradually during unloading, whereas that of the model changed with the sign of resistance, reflecting the properties of Takeda, Takeda-Slip, and Origin-Oriented hysteresis models. The model and structure showed some pinching behaviour. The pinching behaviour of the model was caused by Takeda-Slip and Axial-Stiffness hysteresis models.

Local Deformations

Computed local deformations of typical members were compared with the observed deformations so as to examine the reliability of the analysis method.

Beam End Rotation -- Rotation at beam ends was determined from the longitudinal deformation measurements by two displacement gauges, one placed above the slab face and the other placed below the beam. The observed base shear-beam end rotation relations of a sixth floor beam, connected to the shear wall, is shown in Fig.17 with the calculated relations. The observed rotation was approximately 60 to 70 percent of the calculated amplitudes because the beam end rotation was measured over one-half effective depth of the beam, whereas the rotation was calculated over one-half span length. General shapes of the observed and calculated base shear-beam end rotation relation curves were similar.

Although positive and negative amplitudes of overall structural displacement were comparable, both observed and calculated beam-end rotation amplitudes were consistently greater in negative loading direction (loaded from left to right), which was probably associated with a significant upward movement of the beam-wall node due to the elongation of the tensile boundary column of the first-story shear wall. Comparing the rotation amplitudes at the two ends of the beam, negative deformation amplitudes were comparable, whereas the positive deformation at the wall end was approximately 1.3 times larger than that at the exterior column end. This may be explained by noting high negative and low positive flexural resistances of the beam; i.e., under positive loading, the wall-end of the beam was subjected to positive bending and yielded much earlier than the exterior column-end subjected to the negative bending. In addition, the lateral deformation of the exterior column reduced the nodal rotation at the exterior beam-column joint, resulting in a smaller beam-end rotation.

Figure 18 shows the beam-end rotations of a sixth floor exterior beam in Frame A. Again, the observed beam-end rotation amplitudes were smaller than the calculated amplitudes, but the general hysteresis shapes were similar.

Column Axial Deformation -- The base shear and axial deformation of the first-story boundary columns of the shear wall are studied in Fig.19. The boundary columns were measured to elongate as much as 44 mm in the first story, whereas the maximum compressive deformation reached only 5 mm. This elongation of the tension-side boundary column caused a large vertical displacement at the upper wall-beam joints. General deformation amplitudes and hysteresis shapes of the analytical model, expressed as the deformation of the outer truss elements, agreed reasonably well with those of the test structure. The computed axial deformation was larger.

Response at Maximum Displacement

It is important from design point of view to estimate possible force amplitudes and deformation ductility factors at various critical sections of the test structure at maximum deformation. However, member forces could not be measured in the test, hence, they were estimated by the frame analysis.

Member Forces -- Member forces in Wall-Frame B calculated at maximum structural deformation are shown in Fig.20, including the forces transferred from the transverse beams. Note (a) the first-story shear wall carried 58 percent of the calculated base shear (=4,170 kN), (b) the shear force carried by the first-story wall was smaller than that carried by the second-story wall, (c) the shear forces transferred from the transverse beams connected to the tension-side boundary columns were approximately 2.8 times larger than those from the other side transverse beams, and (d) beam moments of a span were comparable from the second through seventh flool levels.

Member Ductility -- Ductility factors of members were defined in the analysis as a ratio of maximum deformation amplitude to the calculated yield amplitude. Figure 21 shows the distribution of ductility factors at the maximum structural deformation for Frames A and C, and Wall-Frame B. In open Frames A and C, almost all beam ends yielded except those at the roof level. Ductility factors of beams ranged from 0.8 to 1.4 at the left end (subjected to negative bending), and from 2.2 to 4.7 at the right end (subjected to positive bending). Since the rotation amplitudes at the left and right ends of a beam were comparable, the difference in ductlity factors was caused by the difference in the positive and negative yield rotations (Table 3). Ductility factors at the same end (left or right) of the beams decreased with height, whereas those of columns increased with height. A beam end rotation appeared to be inversely related to the column end rotation of the joint; for example, the beam end rotations were larger at the right exterior joints where the column rotations were smaller.

In Wall-Frame B (Fig.21.b) all beams yielded. The beam end ductility factors were relatively uniform along the height; 1.4 to 1.7 at left exterior beam ends, 4.1 to 4.7 at beam ends immediately left of the wall, 3.2 to 3.4 at beam ends immediately right of the wall, and 6.6 to 7.9 at right exterior beam ends. The beam end rotations were generally larger in the right exterior span than that in the left exterior beams, attributable to the large vertical elongations of the tensile boundary columns.

ANALYSIS OF TEST SPD-2

The maximum roof-level displacement during the second test run (Test SPD-2) reached 1/670 of the total structural height, or 33mm. The calculated response waveforms and equivalent SDF hysteresis relations are examined below.

Response Waveforms -- Observed and calculated roof-level displacement and base shear waveforms are compared in Fig.22.a. The analysis indicated that the test structure responded elastically up to 1.5 sec. The calculated response waveforms (solid lines) were in a good agreement with the observed (broken lines) in the first 2.5 sec, and then significantly deviated from the observed. The maximum roof-level displacement of 32.9 mm was observed at 2.03 sec, while that of 36.5 mm was calculated at 2.06 sec. The maximum base shear of 2,200 kN was measured at 2.01 sec, slightly prior to the maximum deformation, whereas that of 2,150 kN, slightly smaller than the observed, was calculated at 2.06 sec. The calculated residual displacement at the termination of the base motion was so small that the free vibration response was not excited in the analysis.

Equivalent Hysteresis Relation -- Observed and calculated roof-level displacement vs. base shear relations are compared in Fig.22.b. Note that the two curves are similar. However, a careful inspection revealed that the calculated stiffness and resistance (solid lines) were generally lower than the observed (broken lines). The calculated stiffness in a small amplitude excursion was lower, which might be a major cause to create the discrepancy in the two waveforms after 2.5 sec.

ANALYSIS OF TEST SPD-4

After test SPD-3, the roof-level displacement during test SPD-4 reached as large as 1/64 of the total height, or 342 mm. The analysis was carried out continuously for tests SPD-2, SPD-3, and SPD-4. Calculated and observed response waveforms and equivalent SDF hysteresis relations are compared below.

Response Waveforms -- Observed and calculated roof-level displacement and base shear waveforms are compared in Fig.23.a. Note a good agreement of the two waveforms over the entire duration of the test. Maximum roof-level displacement reached 342 mm at 4.36 sec during the test, while that of 391 mm was calculated at 4.33 sec. Observed maximum base shear of 4,310 kN was attained at 2.52 sec, much before the maximum displacement; the base shear at the maximum displacement was observed to be 4,250 kN, almost the same as the maximum amplitude. The maximum base shear of 4,540 kN, slightly larger than the observed, was calculated at 4.33 sec.

Equivalent SDF Hysteresis Relation -- Observed and calculated roof-level displacement vs. base shear relations are compared in Fig.23.b. The two hysteresis curves showed a light pinching behaviour at low stress levels. As expected from the good agreement in the response waveforms, the two hysteresis curves agreed well.

CONCLUDING REMARKS

A nonlinear dynamic analysis method was used to simulate the behaviour of the full-scale seven-story reinforced concrete structure tested using equivalent single-degree-of-freedom pseudo-dynamic earthquake response test procedure. The method utilized three different member models for (a) beams and columns, (b) shear walls, and (c) transverse beams, and four hysteresis models for elements of member models: (a) Takeda Hysteresis model, (b) Takeda-Slip hysteresis model, (c) Axial-Stiffness hysteresis model, and (d) Origin-Oriented model. The method to determine the stiffness properties were outlined on the basis of material properties and the structural geometry in addition to the past experience and engineering judgement.

A good correlation was reported between the observed and calculated response when the structure responded well in an inelastic range and a set of model parameters were properly chosen. However, it was felt more difficult to attain a good correlation when the structural response reached barely yielding. The method of nonlinear dynamic analysis of a reinforced concrete building can be made significantly reliable not only to outline the overall structural behaviour, but also to describe the local behaviour, provided proper sets of parameters are chosen for the hysteresis and member models.

ACKNOWLEDGEMENT

The writer wish to express a sincere gratitude to Dr. H. Umemura, Co-Chairman, and Dr. M. Watabe, Japanese Technical Coordinator, U.S.-Japan Cooperative Research Program Utilizing Large Scale Testing Fcilities, for providing the writers with an opportunity to actively participate in the reasearch project. Mr. S. Okamoto, Dr. S. Nakata, and Dr. M. Yoshimura, Building Research Institute, gave the detail description of the test structure, testing method and test results of the structure.

The computation reported herein was carried out on the computer system of the University of Tokyo Large Size Computer Center.

REFERENCES

1. Architectural Institute of Japan, "AIJ Standard for Structural Calculation of Reinforced Concrete Structures (in Japanese)", revised in 1982.

2. Eto, H., and T. Takeda, "Elasto-Plastic Earthquake Response Analysis of Reinforced Concrete Frame Structure (in Japanese)", Proceedings, Architectural Institute of Japan Annual Meeting, October 1977, pp.1877-1878.

3. Giberson, M.F., "Response of Nonlinear Multi-Story Structures subjected to Earthquake Excitation", EERC Report,

Earthquake Engineering Research Laboratory, California Institute of Technology, Pasadena, California, 1967.

4. Hiraishi, H., S. Nakata, Y. Kitagawa, and T. Kaminosono, "Preliminary Component Tests in Japan", to be published in ACI Special Publication on U.S.-Japan Cooperative Research Program, Phase-1: Reinforced Concrete Structures.

5. Hirosawa, M., "Past Experimental Results on Reinforced Concrete Shear Walls and Analysis on Them (in Japanese)", Kenchiku Kenkyu Shiryo No.6, Building Research Institute, Ministry of Construction, 1975.

6. Kaminosono, T., Y. Kitagawa, S. Okamoto, and M. Yoshimura, "Testing Procedures and Preliminary Test Results", to be published in ACI Special Publication on U.S.-Japan Cooperative Research Program, Phase-1: Reinforced Concrete Structures.

7. Nakata, S., S. Otani, T. Kabeyawsawa, Y. Kai, and S. Kimura, "Tests of Reinforced Concrete Beam-Column Assemblages -- U.S.-Japan Cooperative Research Program-", Report submitted to Joint Technical Coordinating Committee, U.S.-Japan Cooperative Research Program, Building Research Institute and University of Tokyo, 1980.

8. Okamoto, S., J.K. Wight, S. Nakata, M. Yoshimura, and T. Kaminosono, "Testing; Repair and Strengthening; Retesting", to be published in ACI Special Publication on U.S.-Japan Cooperative Research Program, Phase-1: Reinforced Concrete Structures.

9. Otani, S., "Nonlinear Dynamic Analysis of Reinforced Concrete Building Structures", Canadian Journal of Civil Engineering, Vol. 7, No.2, April 1980, pp. 333-344.

10. Otani, S., "Effectiveness of Structural Walls in Reinforced Concrete Buildings during Earthquake," Civil Engineering Studies, Structural Research Series No. 492, University of Illinois at Urbana- Champaign, June 1981.

11. Penzien, J., H. Umemura, M. Watabe, and R. Hanson, "Background of U.S.-Japan Cooperative Research Program", to be published in ACI Special Publication on U.S.-Japan Cooperative Research Program, Phase-1: Reinforced Concrete Structures.

12. Sugano, S., "Experimental Study on Restoring Force Characteristics of Reinforced Concrete Members (in Japanese)", Doctor of Engineering Thesis, University of Tokyo, 1970.

13. Takeda, T., M.A. Sozen, and N.N. Nielsen, "Reinforced Concrete Response to Simulated Earthquakes", Proceedings, ASCE, Journal of Structural Division, Vol.96, No.ST12, December 1970, pp.2557-2573.

14. Tomii, M., and Y. Osaki, "Shearing Resistance of

Reinforced Concrete Bracing Walls", Transactions, Architectural Institute of Japan, Number 51, September 1955, pp. 83-95.

15. Yoshimura, M., S. Nakata, and S. Kurose, "Data Analysis", to be published in ACI Special Publication on U.S.-Japan Cooperative Research Program, Phase-1: Reinforced Concrete Structures.

16. Wight, J.K., S. Nakata, and T. Kaminosono, "Construction and Design Code Comparison for the Full Scale Structure", to be published in ACI Special Publication on U.S.-Japan Cooperative Research Program, Phase-1: Reinforced Concrete Structures.

Table 1 Properties of Reinforcement

Bar Size	Nominal Area mm^2	Yield Strength MPa	Strain Hardening Strain	Tensile Strength MPa	Fracture Strain
D10	71	380	0.018	556	0.17
D16	199	378	0.019	560	0.18
D19	287	353	0.017	561	0.20
D22	387	346	0.012	563	0.21

Table 2 Properties of Field Cured Concrete

Story	Test Age (days)	Compressive Strength MPa	Strain at Compressive Strength	Tensile Strength MPa
7	67	18.5	0.0019	1.29
6	87	14.1	0.0019	1.30
5	98	28.9	0.0019	2.31
4	111	28.4	0.0023	2.28
3	119	26.9	0.0023	2.24
2	132	28.6	0.0024	2.41
1	145	28.3	0.0022	2.37

Note : Average Elastic Modulus = 23,200 MPa.

Table 3 Skeleton Moment-Rotation Relation of Beams

Stiffness Properties	(unit)	Negative Bending	Positive Bending
Crack Moment	(kN-m)	89	41
Crack Rotation	($10^{-3} \times \ell$ rad)	1.08	0.50
Yield Moment	(kN-m)	431	96
Yield Rotation	($10^{-3} \times \ell$ rad)	1.08	0.52

Note : 1) Elastic rotation included in rotation.
2) ℓ = span length of beam.

Table 4 Initial Axial Loads (kN) in Vertical Members

(a) Independent Column

Story	C_1*	C_1'	C_2	C_3*	C_3'
7	82	82	138	106	106
6	212	182	272	296	237
5	342	282	408	485	370
4	470	382	542	676	502
3	600	482	678	866	634
2	728	582	812	1056	766
1	862	687	949	1254	905

Note : 1) Column notation given in Fig. 1.
2) * Loading-side column carried additional weight of actuators and loading beams.

(b) Shear Wall and Boundary Columns

Story	Boundary Column	Wall Panel
7	148	106
6	282	229
5	417	353
4	551	477
3	685	600
2	820	724
1	956	855