EXPERIMENTAL MODELING AND ANALYSIS OF THREE ONE-TENTH SCALE REINFORCED CONCRETE MODELS OF THE BRI TEST STRUCTURE

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SUMMARY

Results of one-tenth scale models tsted on an earthquake simulator at the University of Illinois are described. The prototype was a full-scale structure tested at the Building Research Institute (BRI) in Tsukuba, Japan. Earthquake records used in the simulations were the Miyagi Ken-Oki 1978 (N-S) record and the Taft 1952 (E-W) record.

INTRODUCTION

There is need to test full-scale structures to determine the relationship between full-scale tests and results of small-scale tests, component tests, and analytical models. For this purpose, a cooperative research project between the United States and Japan was initiated in 1977. Experimental studies were conducted on full-scale, one-fifth scale and one-tenth scale structures. Component tests were also run.

The full-scale structure was tested pseudo-dynamically at the Building Research Institute (BRI) in Tsukuba, Japan. All of the small-scale models were tested on earthquake simulators. At the University of California (Berkeley), a one-fifth scale replica of the BRI structure was tested. It contained floor slabs and also beams and walls transverse to the loading direction. The one-tenth scale structures, tested at the University of Illinois, were two-dimensional models.

OUTLINE OF ONE-TENTH SCALE MODEL TESTS

The three one-tenth scale models, designated NS1, NS2, and NS3, were composed of individual planar systems representing the lateral load resisting elements of the full-scale structure. The models contained two exterior frames and one interior frame with a wall. There were no floor slabs in the models. Figure 1 shows the layout of the large-scale and one-tenth scale structures. The total height of the models was 2150 mm with a first story height of 350 mm. All other stories were 300 mm high. The nominal dimensions for the beams, columns, and wall in the one-tenth scale models were 30 mm x 50 mm, 50 mm x 50 mm, and 20 mm, respectively.

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Material properties of the structure are listed in Table 1. Bright basic annealed wire was used for the reinforcement in the models. After fabrication of the reinforcement cages was completed, the reinforcement was rusted to develop deformations on the surface of the wires for better bonding with the concrete. The reinforcement was then cleaned and placed in forms on a horizontal casting plate. The frames were cast with small-scale concrete made of cement, fine sand, coarse sand and water.

The first model, NS1, contained beam and column reinforcement which was equivalent to that of the full-scale structure (Fig. 2). Structures NS2 and NS3 had additional beam steel to account for the contribution of the slab steel in the full-scale structure. The third model, NS3, had a slightly higher concrete strength in the wall-frame. Longitudinal reinforcement in the small-scale structures was selected to simulate the reinforcement ratio and resisting moment of the full-scale structure. Shear reinforcement was provided by a continuous spiral to avoid a shear failure. Also, anchorage details were changed to reduce the possibility of problems related to development.

Story masses (nominally 444 kg per story including the harness) were placed on the models to obtain the desired fundamental frequency. The harness for the masses provided coupling among the three frames so that the corresponding nodes in each frame would have the same displacement in any direction with negligible rotational resistance.

In addition to the increased mass, the time axis of the input motions was compressed by a factor of five to maintain the ratio of the natural frequency of the large and small structures to the frequency content of the input motions.

Base motions for the small-scale structures were in one horizontal direction only. This was also the case for the load configuration applied to the full-scale structure. The earthquake records were chosen to correspond to those applied to the other models. The Miyagi Ken-Oki (1978) record was chosen as the primary earthquake since it was to be used by the University of California (Berkeley) for their tests. The test sequence for the one-tenth scale models is listed in Table 2.

EXPERIMENTAL OBSERVATIONS

A comparison of the response histories of the three one-tenth scale structures indicated good correspondence when the structures were subjected to similar base motions (Ref. 1). Selected response histories of one of the models, Structure NS2 (Runs 1 and 4), are shown in Figure 3.

The initial portion of the Miyagi (1978) base acceleration record contains a very low amplitude motion. In Run 1, the response of NS2 in this region confirmed the value of approximately 10.7 Hz obtained for the natural frequency of the structure from a free vibration test. The frequency of the structure decayed throughout the test to a value of approximately 5.0 Hz. Every increase in displacement in Run 1 corresponded to an increase in base shear and moment indicating the structure had not yet reached its full capacity. Cracking was light. There was only one major flexural crack in the wall.

The magnitude of the base acceleration in Run 4 was approximately three times that in Run 1. As expected, the structure was much more flexible in Run 4. The frequency decayed to less than 3 Hz near the end of the test run. In Run 4 the base moment and shear reached their maximum values before the displacement reached its maximum. Rocking about the major flexural crack created in Run 1 was observed. A visual inspection of the structure revealed that reinforcement in the shearwall had fractured. These observations indicated the structure had reached its maximum shear and moment capacity in the final test run.

The response of the structure for all of the test runs was dominated by the first mode. This can be seen more clearly in Figure 4 which shows the distribution of the displacements over the height of the structure at particular points in time. This was essentially true of the acceleration records also. Higher mode components were visible only at the lowest levels which tend to reflect the base motion.

A summary of the maximum response values for all three structures is listed in Table 3.

DISCUSSION OF EXPERIMENTAL RESULTS

Limit analysis was used to calculate the maximum base shear capacity of the structures before testing. The structures containing the heavier beam reinforcement (NS2 and NS3) were estimated to have a forty percent larger capacity than the lighter reinforced structure (NS1).

The structures had nearly twice the capacity calculated from two-dimensional analysis. The increase in base shear and moment capacity of the structure was due to a three-dimensional effect. The harness, used to attach the additional story masses to the structure, acted as a stiff floor system. Vertical displacements due to rocking of the wall were transmitted to the exterior frame by the harness. The uplift created a tension in the exterior frame columns adjacent to the wall which increased the capacity of the structure. This is similar to the effect of the floor system observed in the full-scale structure.

Three-dimensional effects in the models increased the capacity to such an extent that the variation of beam reinforcement did not influence the response as much as expected.

A plot of the base moment capacity of the three one-tenth scale models versus top-story displacement is shown in Figure 5. At small displacements the three structures had essentially the same capacity. After yielding of the beams occurred, the base moment capacity of the heavily reinforced models (NS2 and NS3) increased more than the lightly reinforced model (NS1), as expected. At most this difference amounted to ten percent.

CONCLUSIONS

The pseudo-dynamic tests conducted on the full-scale structure were based on the assumption that the structure would behave predominantly in the first mode. The dynamic response of the small-scale structures confirmed this assumption.

The three-dimensional effect (the wall coupled with adjacent columns in the exterior frames) increased the shear and moment capacity of the small-scale structures significantly. This effect was also observed in the full-scale structure due to the floor system and transverse beams.

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Table 1 Material Properties

Steel Properties

Location	Wire Gage	Diameter mm	Strain Rate 1/sec	Yield Stress** kgf/mm^2	Strength** kgf/mm^2
Top of Beams (NS2 & NS3)	No. 7	4.5	0.001	37.1 ± 1.1	43.0 ± 0.9
Columns	No. 13	2.3	0.001	39.2 ± 1.1	42.5 ± 0.9
			0.005	40.7 ± 1.2	43.4 ± 1.5
Beams and Wall	No. 15	1.8	0.001	42.7 ± 0.7	45.7 ± 0.6
			0.005	42.9 ± 0.7	46.5 ± 0.6
Transverse Steel	No. 16	1.6	0.001	79 ± 1.2	85 ± 3.1

**Mean ± standard deviation based on ten samples each.

Concrete Properties

Test Structure	Frame	Age at Testing days	Initial Modulus ton/cm^2	Compression Strength* kgf/cm^2
NS-1	North	286	188	313 (22)
	Center	244	195	307 (23)
	South	324	215	361 (15)
overall NS-1			199	327 (31)
NS-2	North	93	191	256 (30)
	Center	163	194	307 (29)
	South	178	182	291 (16)
overall NS-2			189	284 (34)
NS-3	North	217	193	259 (26)
	Center	135	217	392 (28)
	South	302	178	227 (18)
overall NS-3			196	306 (78)

*Mean (Standard Deviation)

Table 2 Test Sequence

	Overall Drift Ratio* Target %	Overall Drift Ratio* Attained %	Max. Base Acceleration g	Spectrum Intensity** (β = 10%) mm	Ground Motion
NS1 - Run 1	0.75	0.82	0.59	132	Miyagi Ken-Oki 1978 (N-S)
NS2 - Run 1	0.75	0.70	0.59	131	"
NS3 - Run 1	0.75	0.63	0.49	121	"
NS2 - Run 2	1.00	1.40	1.00	223	"
NS3 - Run 2	1.00	1.10	0.82	188	"
NS2 - Run 3	1.00	1.40	1.60	180	Taft 1952 (E-W)
NS3 - Run 3	1.00	1.10	1.50	201	"
NS1 - Run 2	>2.00	2.20	1.80	424	Miyagi Ken-Oki 1978 (N-S)
NS2 - Run 4	>2.00	2.60	1.50	340	"
NS3 - Run 4	>2.00	2.30	1.50	306	"

*Overall drift ratio is the top story displacement divided by the height from the base to the top story.

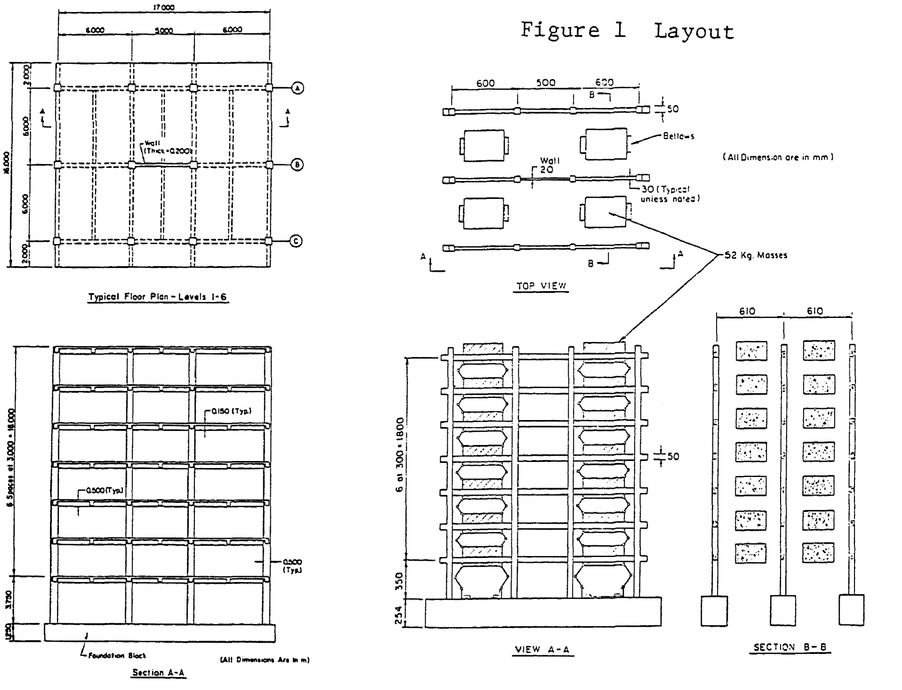
**Spectrum intensity is the integral of the velocity response spectrum taken over the range of structural vibration periods from 0.02 to 0.5 sec.

Note: Free vibration tests are run before and after each earthquake simulation.

Table 3 Maximum Values of Response

	Base Acceleration g	Ground Motion	Top-Level Displacement mm	Top-Level Acceleration g	Base Shear kgf*10^3	Base Moment kgf-m
NS1 - Run 1	0.6	Miyagi Ken-Oki 1978 (N-S)	18	1.6	3.06	4510
NS2 - Run 1	0.6	"	15	1.7	3.20	4670
NS3 - Run 1	0.5	"	14	1.5	2.80	4060
NS2 - Run 2	1.0	"	29	2.4	3.51	5500
NS3 - Run 2	0.8	"	23	2.2	3.46	5210
NS2 - Run 3	1.6	Taft 1952 (E-W)	29	2.0	3.16	4580
NS3 - Run 3	1.5	"	24	2.3	3.45	4510
NS1 - Run 2	1.8	Miyagi Ken-Oki 1978 (N-S)	47	2.6	3.94	5630
NS2 - Run 4	1.5	"	56	3.0	4.29	6110
NS3 - Run 4	1.5	"	50	3.1	3.95	6260

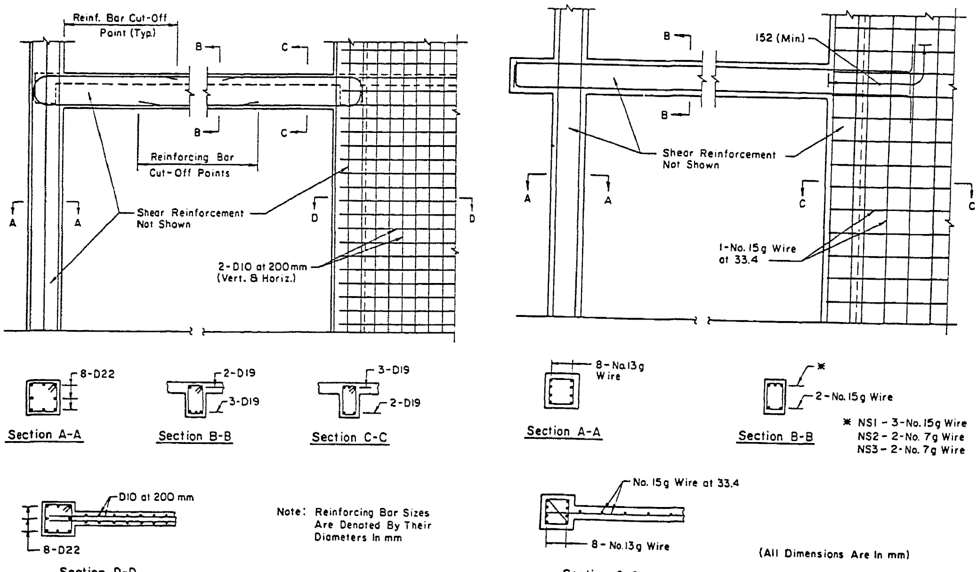
Figure 1 Layout



Full-Scale Model

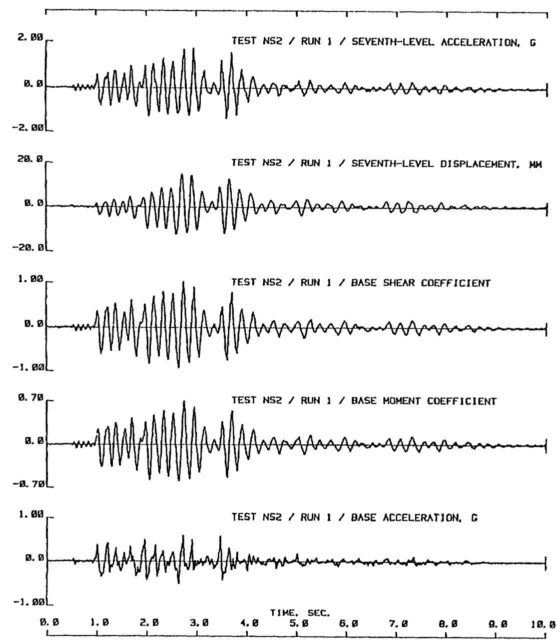
One-Tenth Scale Model

Figure 2 Reinforcement Details

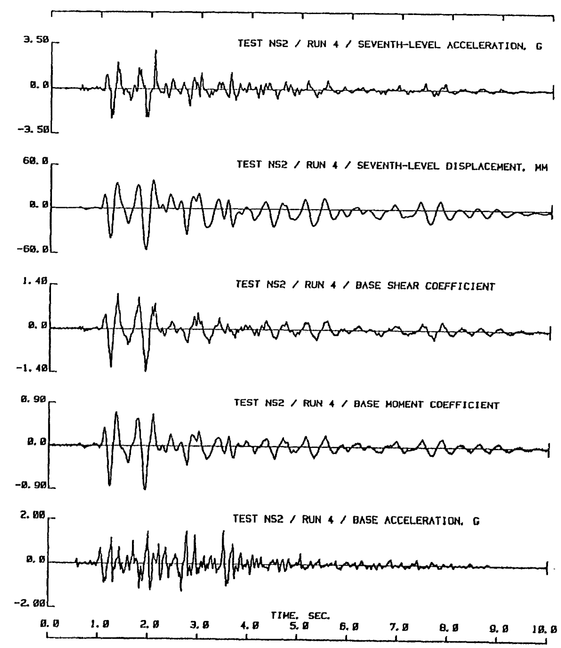


One-Tenth Scale Model

Full-Scale Model



a) NS2 Run 1



b) NS2 Run 4

Figure 3 Response Histories

Figure 4 Distribution of Displacement Response (NS2 Run 1)

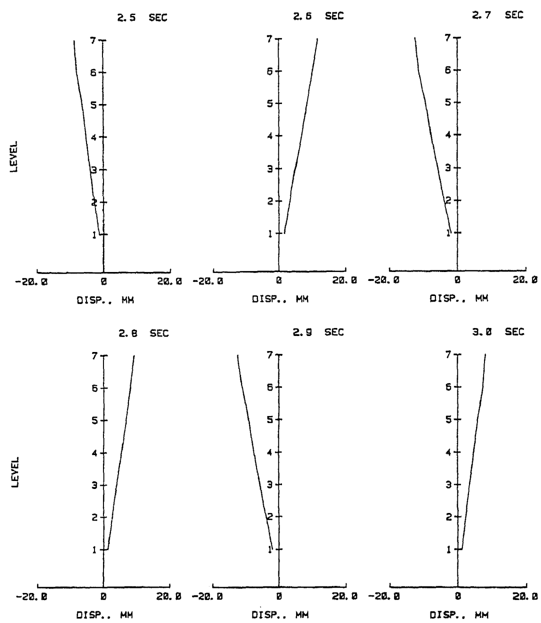


Figure 5 Base Moment vs. Top-Story Displacement