RESPONSE AND FAILURE OBSERVATION OF WEAKLY DESIGNED STEEL STRUCTURE MODELS

(Part I) A Progress Report —— An Outline of the Project and the Preliminary Computer-Actuator On-line Analysis

by

Koichi TAKANASHI (I), Kenichi OHI (II), and Xiaohang GAO (III)

1. INTRODUCTION

It is generally recognized that response simulating techniques, such as shaking table test and computer-actuator on-line test, provide an effective means to observe the response of a steel building and to verify and improve the practical procedure of earthquake resistant design. There remain some difficulties, however, in the case that more complicated phenomena have to be dealt with as follows:
(1) the interaction between the ground and the building, and
(2) the inelastic responses, including translation, rotation, and rocking, caused by multi-directional seismic excitations.

Observing the actual response of existing steel buildings in use is one of the effective approaches to investigate the above phenomena, but the observation of considerable damage would be scarcely expected. Another approach is to construct a structure model weaker than existing buildings and to observe its response during a moderate earthquake.

In this attempt the project of response and failure observation using 'weak' steel structure models has been carried out in the Chiba Experiment Station, Institute of Industrial Science, Univ. of Tokyo, since August in 1983. One of the models is designed so that it may be slightly damaged by a moderate earthquake at Intensity IV to V (Japan Meteorological Agency Scale, less than 80 gals in the ground acceleration) and it may collapse at Intensity V (about 80 to 250 gals).

This paper describes an outline of this project and the preliminary response analysis on the model using computer-actuator on-line system.

2. AN OUTLINE OF THE PROJECT

Weakly Designed Steel Structure Models

Three models were constructed; two models were installed on the actual ground and one was fixed to the testing floor in the laboratory. The outline of each model is summarized as follows:

(I) Professor, (II) Research Associate, (III) Graduate Student of Institute of Industrial Science, University of Tokyo

No. 1 (Installed on the Ground)

A three-story moment resistant frame composed of H-shaped columns and girders; this model is designed slightly weaker than the design practice of Japan, and it is expected to remain elastic during a moderate earthquake.

No. 2 (Installed on the Ground)

A three-story braced frame composed of H-shaped columns and girders and rectangular-section tension bars; the yielding of the tension bar will occur at about 80 gals in the ground acceleration, and this model is expected to collapse under seismic input less than 200 gals.

No. 3 (Fixed to the Testing Floor)

This model has almost the same specifications of columns and tension bars as those of the No.2 model. Its response to El Centro NS excitation is simulated by the computer-actuator online system, which is reported herein.

The reinforced concrete basements of No.1 and No.2 models were installed directly on the surface of Kanto loam after the top soil (50 cm thick) were removed. The shapes and the dimensions of No.1 and No.2 models are shown in Figs. 1 and 2, and the important parameters of the two models are summarized on Tables 1 and 2.

Instrumentation

Various types of transducers were installed on each model to measure the following response data:

(1) accelerations of each floor, 3x3 components.
(2) accelerations of RC basement, 3 components.
(3) inter-story displacemnts including rotation, 4x3 components.
(4) flexural strains of 1st story columns and axial strains of tension bars, 32 components.

Additionally, 3x2 components of the underground motions (1 meter and 40 meters deep) are simultaneously recorded. The data acquisition is automatically started once one gal of the underground acceleration is sensed at 40 meters deep, and the data are converted into digital form with the sample time of five milliseconds.

3. RESPONSE OBSERVATION IN ELASTIC RANGE

The response observations were experienced more than twenty times since August in 1983 till May in 1984. Six cases are summarized on Table 3. So far as shown on Table 3, the both models remain elastic. Before May in 1984 the No.2 model was strengthened and stiffened by additional the bracings so as not to collapse before confirming the reliability of instrumentation.

Figs. 3 and 4 show the time histories and the Fourier amplitude spectra, respectively, for the acceleration records in the weak-axis direction of No.1 model, which were recorded on 6 March in 1984. It is found that the frequency-domain characteristics of the two underground accelerations, 1 meter and 40 meters deep, clearly differ from each other.

The Fourier amplitude spectra of the floor responses have three well-separated peaks, and the frequencies at the peaks can be regarded as the natural frequencies of No.1 model. The natural periods derived from the Fourier spectra are compared with the calculated ones on Table 4. In the calculation of natural period the following assumptions are made:

(1) The flexural and shear deformations of H-shaped columns are considered, while the deformations of girders and RC slabs are ignored.

(2) The deformation of columns are assumed to take place between the surfaces of the RC slabs, and the column bases are assumed to be completely fixed.

(3) The effect of secondary bending moment caused by the weight of the model, so-called p- Δ effect, is considered.

(4) The deformations of the RC basements and the ground are ignored.

There are two principal directions in the elastic stiffness of the model to lateral loads, corresponding to the principal axes, weak-axis and strong-axis, of the H-shaped column section. The stiffness of the model in each directions can be calculated under the above assumptions, and the natural periods are obtained from the eigen-value analysis on a planar vibrational system. As shown on Table 4, the calculated periods slightly underestimate the periods derived from the Fourier spectra.

Furthermore, it is found that the complex frequency response function or the transfer function of the models can be evaluated from the records, using the finite Fourier transformation techniques as follows:

$$Hi(f) = \ddot{X}i(f) \,/\, \ddot{X}o(f)$$

where f : discrete frequency in the finite Fourier transformation

$Hi(f)$: transfer function for the i-th floor acceleration

$\ddot{X}i(f)$: finite Fourier transform of the i-th floor acceleration

$\ddot{X}o(f)$: finite Fourier transform of the acceleration of the RC basement

The transfer function for the 3-rd floor acceleration in the weak-axis direction of No.1 model is evaluated from the records observed on 6 March in 1984, and its amplitude and phase angle are shown in Fig.5. The broken line in Fig.5 shows a theoretical transfer function of a planar 3-DOF vibrational system, which approximates the evaluated one. An appropriate smoothing technique should be applied to these identification techniques using finite Fourier transformation.

3. PRELIMINARY COMPUTER-ACTUATOR HYBRID ANALYSIS

The No.3 model installed on the testing floor has almost the same specifications of columns and tension bars as the No.2

model. It must be noted, however, that there remains several differences in the mechanical characteristics between the two models:

(1) The influence of the ground around the RC basements are ignored in the No.3 model.

(2) While the columns of No.3 model have almost no axial load, the columns of No.2 model are axially compressed to about 40 % of the yield axial force.

(3) When Θ denotes the angle between the axis of tension bar and the horizontal line, cosine Θ for No.2 model equals to 0.87 while the value for No.3 model is 0.71 to 0.7. Therefore the contribution of the bracing used in No.3 model to the total strength or stiffness is reduced to 80 % or 70 % of the contribution of that used in No.2 model, repectively.

The details of the computer-actuator on-line system used herein have been already reported in Refs.[1] and [2]. In this system a loading test is carried out on a structural model to measure the restoring force used in the step-by-step numerical integration of equation of motion.

This system was applied to simulate a uni-directional response in the weak-axis direction of No.3 model, as shown in Fig.6. The assumed weight of each floor is set to 15 tons smaller than the actual weight of No.2 model, 18 tons per each floor, in order to compensate for the above mentioned difference in the strength and stiffness(3). As for the difference (2), only the p- Δ effect is considered; the amount of shear proportional to the inter-story displacement is substracted from the measured story shear to obtain the restoring force used in the analysis.

The wave shape of the excitation used herein is the NS component recorded at El Centro in 1940. Two phases of hybrid analyses were carried out, as shown on Table 5.

In the phase I analysis, the peak excitation was set to 130 gals, and the yielding and the bucking of all the tension bars and the inelastic deformation of 1st story columns were observed.

In the phase II analysis the simulation was carried out under the excitation of 160 gals after replacing of braces. It was observed that the inelastic deformations progressively cumulated to a direction, as is often experienced in the case of high axial loads.

Time histories of response displacements and the hysteresis loops are shown in Figs.7 and 8.

5. CONCLUDING REMARKS

(1) An outline of the project of response and failure observation using 'weak' steel structure models has been described herein. Several responses in the elastic range were recorded in 1983, and they proved that the observation system works well and the recorded data are sufficiently reliable. These data can be used in further studies for evaluating the compatibility with various types of soil-structure interaction model and for verifying the response analyses based on these models.

(2) The computer-actuator on-line system was applied to the No.3 model in the laboratory, whose mechanical characteristics were similar to those of No.2 model on the actual ground. So long as the effects of soil-structure interaction are small, it is concluded that the No.2 model will be inelastically damaged by a moderate earthquake at Intensity IV to V and collapse under the seismic input less than 200 gals.

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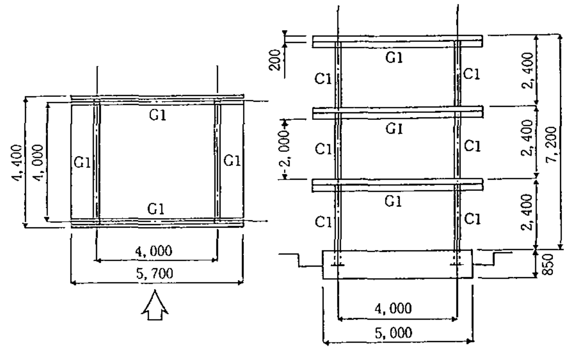


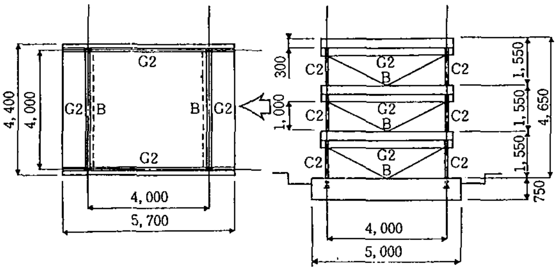
(1) Plan (2) Elevation

Fig.1 Shape and Dimensions of Model No.1

Table 1 Parameters of Model No.1

Stories	3	
Area of Each Floor	25.1 m^2	
Weight of Each Floor	13.2 tons	
Steel Grade	JIS SS 41	
Steel Members Used	C1 : H-125x125x6.5x9	
	G1 : H-200x100x5.5x8	
	Additional * Bracing	L-65x65x6

* Installed during maintenance work and storms



(1) Plan (2) Elevation

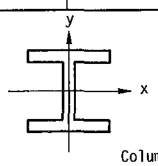
Fig.2 Shape and Dimensions of Model No.2

Table 2 Parameters of Model No.2

Stories	3	
Area of Each Floor	25.1 m^2	
Weight of Each Floor	19.0 tons	
Steel Grade	JIS SS 41	
Steel Members Used	C2 : H-100x50x5x7	
	G2 : H-250x75x6x6	
	B : PL-6x10	
	Additional Bracing	L-65x65x6

Table 3 Summary of Response Observation

Date	Epicenter (Depth in km)	Duration (Magnitude)	No.1 Model Peak Acceleration			No.2 Model Peak Acceleration		
			Direction	Basement	3rd Floor	Direction	Basement	3rd Floor
1983. 8. 8	35°31′N. 139°01′E. (22)	3′19″	x	17.7	----	x	14.4	62.6
12:48:16		(M6.0)	y	15.8	50.7	y	16.3	61.2
1983.10.28	36°12′N. 140°01′E. (60)	1′16″	x	12.4	19.2	x	11.0	50.0
10:50:47		(M5.1)	y	12.0	31.0	y	11.0	51.2
1983.12.30	35°41′N. 140°43′E. (52)	1′40″	x	9.58	31.1	x	11.5	62.2
11:30:53		(M5.4)	y	11.0	34.4	y	11.5	74.3
1984. 1. 1	33°16′N. 136°59′E. (400)	3′43″	x	23.6	72.2	x	23.5	112
18:04:47		(M7.4)	y	28.4	67.4	y	27.3	99.6
1984. 3. 6	29°28′N. 139°08′E. (460)	5′48″	x	24.3	81.4	x	20.1	62.8
11:19:03		(M7.9)	y	30.5	107	y	8.9	----
1984. 4.24	31°00′N. 136°36′E. (450)	1′17″	x	5.17	12.5	x	5.7	31.1
13:13:54		(M6.8)	y	7.39	22.1	y	6.2	26.8



Column Section

x: weak-axis direction of H-shaped column section

y: strong-axis direction of H-shaped column section

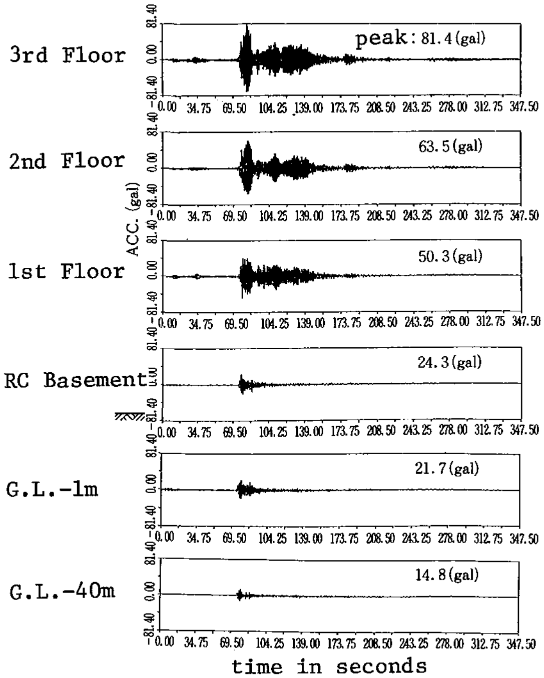


Fig.3 Time Histories of Response Acceleration (Model No.1, 6 March 1984)

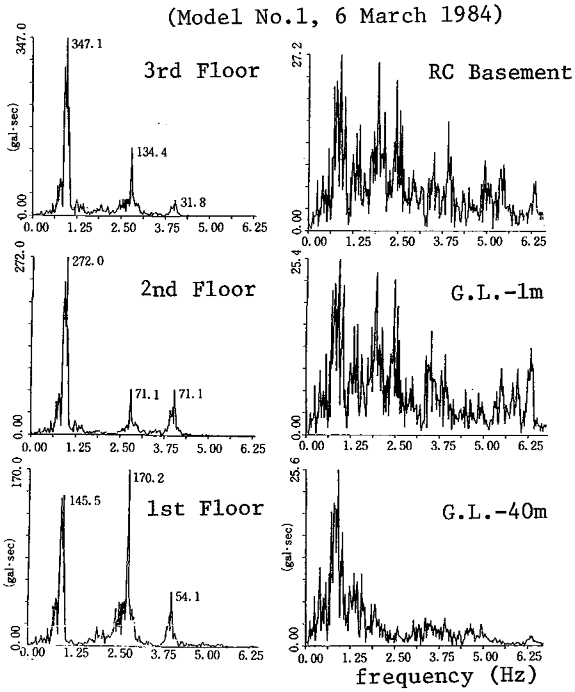
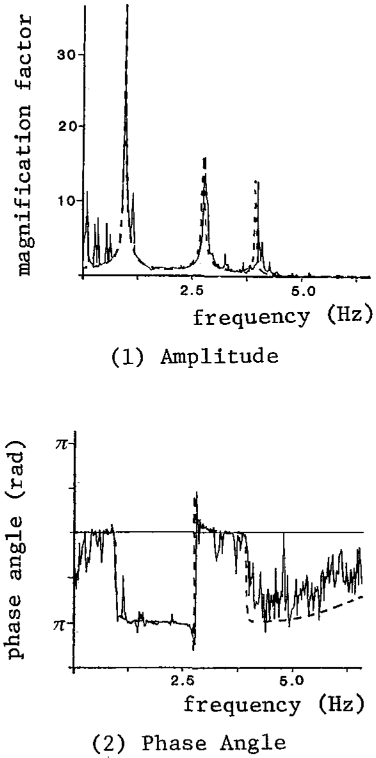


Fig.4 Fourier Amplitude Spectra of Response Acceleration (Model No.1, 6 March 1984)



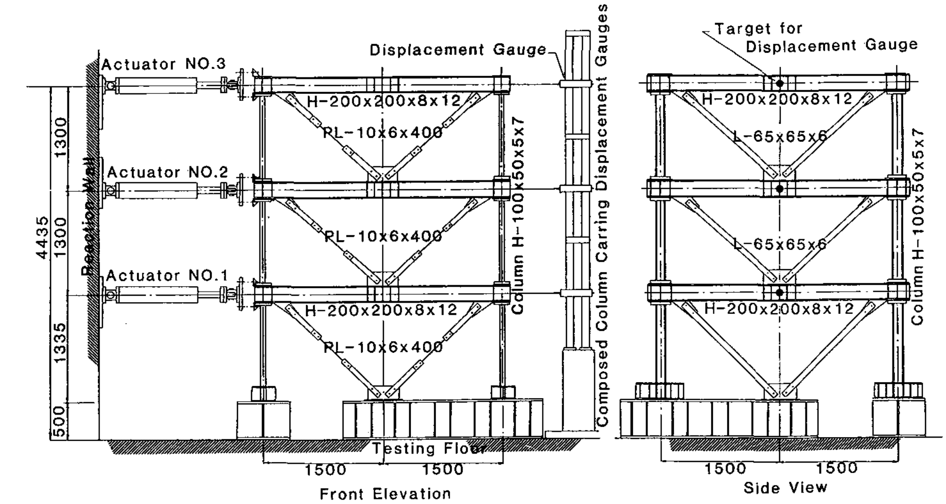
——— Derived from Acceleration Records

----- Approximation

Fig.5 Identification of Transfer Function (Model No.1)

Table 4 Natural Periods of Model No.1

direction	mode	Natural periods (sec)				
		Derived from Fourier Spectra				
		8 Aug.	1 Jan.	6 March	av.	calculated
y	1	0.675	0.669	0.672	0.672	0.604
	2	0.227	0.227	0.226	0.227	0.214
	3	0.159	0.151	0.154	0.155	0.149
x	1	0.984	1.02	1.05	1.02	1.01
	2	0.356	0.356	0.358	0.357	0.357
	3	0.243	0.246	0.250	0.246	0.248



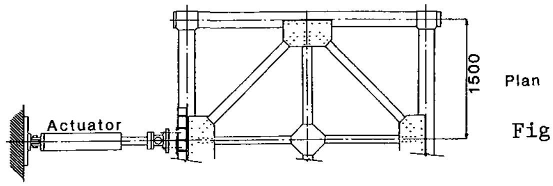
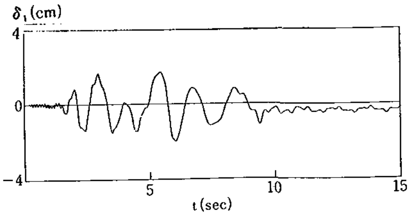
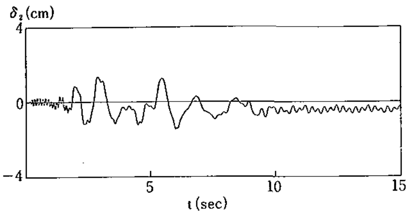


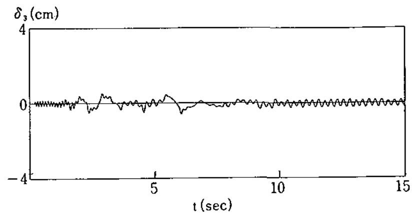
Fig.6 Test Setup (Model No.3)



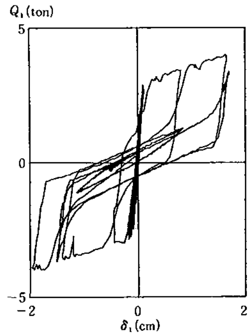
(1) 1st Story Drift



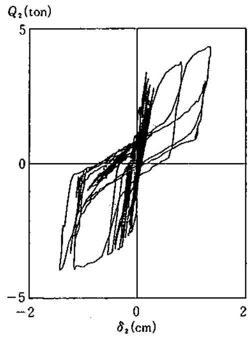
(2) 2nd Story Drift



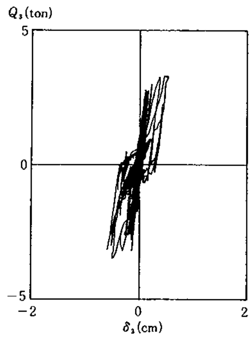
(3) 3rd Story Drift



(4) 1st Story Hysteresis Loop



(5) 2nd Story Hysteresis Loop

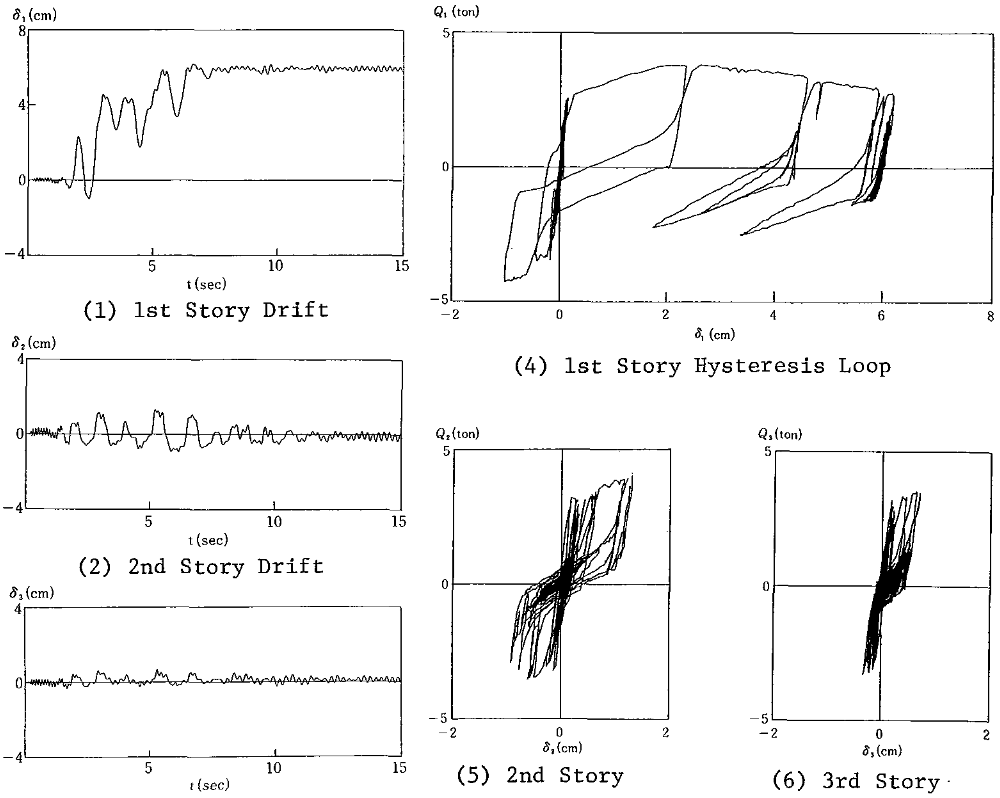


(6) 3rd Story Hysteresis Loop

Fig.7 Phase I Test Results (El Centro NS, max.acc.=130 gals)

Table 5 Summary of Computer-Actuator On-line Tests

Excitation	El Centro NS 10 sec + Free Vibration 10 sec		
Peak Excitation	Phase I		130 gal
	Phase II		160 gal
Assumed Mass	1.53×10^{-2} tcm^{-1} sec^2 per each floor		
Coefficient used in Consideration of P-Δ effect (t/cm)	1st Story		-0.45
	2st Story		-0.30
	3st Story		-0.15
Test Speed	1/100 of Real Time		
Peak Response Displacement (cm)	Phase I	1st Floor	1.98
		2nd Floor	3.38
		3rd Floor	3.98
	Phase II	1st Floor	6.16
		2nd Floor	7.20
		3rd Floor	7.57



(1) 1st Story Drift

(2) 2nd Story Drift

(3) 3rd Story Drift

(4) 1st Story Hysteresis Loop

(5) 2nd Story Hysteresis Loop

(6) 3rd Story Hysteresis Loop

Fig.8 Phase II Test Results (El Centro NS, max.acc.=160 gals)