Experiment and Research on the Response of Steel Model Structures subjected to Impact Horizontal Loading and to Simulated Earthquakes

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鉄骨造１層１スパンフレームの衝撃荷重実験および鉄骨造３層２スパンフレームの振動実験

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概　要

本研究は，従来より進めてきた各種の動的解析法の実験的検討と，鉄骨フレームの弾塑性振動時の諸性状の把握が主目的である。本論文は，これらのまとめの一端として第５回世界地震工学会議に提出したものであって，内容は２部に分かれており，前半では１層鉄骨フレームの衝撃台上での弾塑性振動実験（既報）について，後半では，３層鉄骨フレームの大型振動台での弾塑性振動実験について示す。１層鉄骨フレームの解析には，復元力特性を bi-linear 型にとった質点系解析法および曲線式による質点系解析法を使用し，３層鉄骨フレームの解析には，やはり曲線式による方法と，各部材の弾塑性挙動を考慮したフレーム解析法を使用した。解析にあたり内部減衰および外部減衰の各定数の選択が解析結果に著しく影響を与えることが判明した。

Abstract

Several steel portal frames were tested on a large shock table and on a large earthquake simulator which have been built for this purpose. On the other hand we tried several methods of analysis for tracing of the dynamic behavior, and obtained a good accordance with experimental results. It is realized that inelastic behavior can fully be traced by using Ramberg-Osgood type or bi-linear type hysteresis model.

Notation

M_S: mass
M_C^T: bending moment at the top of column
M_C^B: bending moment at the base of column
M_G^L: bending moment at the left of girder
M_G^R: bending moment at the right of girder
M_P: bending moment at the panel zone
M_Y: yield moment
S: stiffness
h: fraction of critical damping
C_E: external damping coefficient
C_I: internal damping coefficient
$\mathbb{Y}$: acceleration vector
K_C: stiffness of column
K_G: stiffness of girder
K_1: stiffness of inelastic spring in terms of bending moment per unit angle of rotation
α_1: $K_1/2EK_C$ (for column)
θ_C^T: rotation of top of column
θ_C^B: rotation of base of column
θ_G^L: rotation of left end of girder
θ_G^R: rotation of right end of girder
u: relative displacement of column
ε: relative axial deformation of column

$\dot{\mathbb{Y}}$: velocity vector
$\mathbb{Y}$: displacement vector
$\ddot{\mathbb{Y}}_0$: ground acceleration vector
Q_C: shear force in column
Q_G: shear force in girder
N: axial force in column
E: Young's modulus
G: modulus of shear rigidity (panel)
v: relative displacement of girder
γ_p: shear deformation of panel zone
l: length of girder
B: the one half of the width of panel zone
D: the one half of the height of panel zone
t: thickness of panel zone
τ_p: shear stress of panel zone

Preface

This report is divided in two parts. In part I, experimental results of one-bay one-storied steel portal frames under impact loading at their base and elasto-plastic analysis of them are reported. In part II, experimental results of two-bay three-storied portal frame under simulated earthquake motion are reported together with the inelastic frame analysis and with usual shear model analysis.

PART I

1. INTRODUCTION

The main object of this test is to produce plastic condition in the structures in a dynamic loading condition and to investigate their behavior. Such kinds of the experiments of one-storied steel frames were done with use of shock table and the inelastic behavior was analized by assuming lumped mass system with Ramberg-Osgood or bi-linear hysteresis model.

2. SPECIMENS AND TESTS

Four steel specimens with same shape and material property were made as indicated in Fig. 2 and in Table 1. Every specimen was designed to fail at the columns which were *H*-shaped and were welded to the beams. The beams were supposed to be completely stiff because of constraint from the shock table and the steel plates for mass. Three of them were for impact tests and the rest was for static loading test. In the impact test each specimen with 16.6 TON or 21.1 TON weight at the top was fixed at the shock table against which 15 TON pendulum collided. The specimen was subjected to several impacts successively with increasing height of pendulum, in order to investigate the behavior ranging from elastic to plastic condition. Between impact tests, free vibration tests were conducted to observe the change in frequency and damping value.

3. IMPACT TESTING APPARATUS

Outline of impact testing apparatus is shown in Fig. 1. The impulse is obtained by the collision of the pendulum (15–30 TON variable in weight), which is raised up to certain height by the winch and then released, against to the shock table (70 TON in weight) suspended by rods from steel frame.

Intensity and duration time of impulse is controlled by the height of pendulum to be raised up, mass of pendulum and the stiffness of the spring, a kind of shock absorber, provided in front of shock table.

4. MEASURED AND CALCULATED RESULTS

The natural period and damping coefficient obtained during tests are indicated in Table 2. The natural period gets slightly longer, and the damping h increases to 1.5% with increasing excitation. The acceleration and displacement records obtained at the test of SI-III frame are shown in Fig. 6 with calculated values. In the analysis two type of hysteresis were assumed, namely, bi-linear type in case 1 and Ramberg-Osgood type in case 2 which is represented by the formula.

$$\frac{X}{X_Y} = \frac{Q}{Q_Y} + \alpha\left(\frac{Q}{Q_Y}\right)^{\gamma} \qquad \frac{X - X_0}{2X_Y} = \frac{Q - Q_0}{2Q_Y} + \alpha\left(\frac{Q - Q_0}{2Q_Y}\right)^{\gamma} \tag{1}$$

The coefficients in eq(1) are determined with reference to the static loading test results ($\alpha = .07$, $\gamma = 9$). The damping coefficient h is consistently assumed to be 1% with reference to the value at Table 2. As the yield displacement is around 10 mm, the structure SI-III remains in elastic during test D-1 and maximum ductility attained is around 3 during test D-3. The comparison at every test between the measured and calculated results in case 2 shows good coincidence except spike-like maximum acceleration in D-2 and D-3 test. On the other hand, the calculation based on the bi-linear hysteresis model shows slightly larger amplitude than that of experiment at the free vibration part of record. Agreement between the test and calculated results of the other specimen, including the specimen with ductility 6 attained, are almost same as mentioned above.

PART II

1. INTRODUCTION

The object of this experiment is to generate elasto-plastic condition in structures by using earthquake simulator which can feed various ground motion records, and to investigate their behavior. The large earthquake simulator used in this experiment is operated by electro-hydraulic mechanism, has a space of 4.366 m × 5.466 m, and has a capacity of, as much as 15 TON of loading, about 1,000 gal of horizontal acceleration and 5 cm of maximum horizontal displacement. Ground acceleration of EL CENTRO 40 NS stored in magnetic tape is transmitted with desirable intensity to the simulator, and from recorded accelerations, displacements and strains of members, elasto-plastic condition of the frame is observed. On the other hand, from the recorded ground motion on the vibration table, the behavior of the frame is analized and compared with experimental values.

2. TESTING SPECIMEN AND EXPERIMENTAL PROCESS

The specimen, the double frame of 3 stories and two bays, is shown in Fig. 3. The material was SS41 steel and the members were joined by welding. Weight on each floor was 3.965 T (3F), 3.102 T (2F), 3.096 T (1F), including weights which were used to produce axial forces on columns. Testing specimen was designed as column yielding type. To make analysis simple, the specimen was designed such that the effect of the vertical force to the beams, parallel to the vibrational motion, was eliminated as small as possible. Namely, the vertical forces due to masses were transferred through perpendicular beams to columns and the horizontal forces, or inertia forces of the masses, were transferred through thin steel plate slab to beams and then to columns.

In the experiment, at first, the flow of stress and the comparison between shear force and acceleration, were studied. The experiment was continued by changing the intensity of the ground motion. Before and after, these tests by earthquake-like motion, acceleration resonance curves were obtained by inputing sine waves of low amplitude and of varying cycles. From some resonance curves of accelerogram, damping factors of higher modes were difficult to determine. Surveying the results of several resonance tests, some deviation of natural frequencies or damping factors could be observed. At the final stage of experiment, strong resonance test at the 1st natural frequency of the specimen was done. During the test, the top and the bottom of each column at the 1st floor, turned into plastic hinges and were fractured because of fatigue with repeated loading at the portion where the local buckling occured. Response waves of each story are shown in Fig. 7.

Influence of the 1st mode wave excels in every case, but higher mode wave especially affects the accelerogram of the 1st floor.

3. METHOD OF ANALYSIS

Two elasto-plastic analitical methods are adopted in this report. The one is by multi-lumped mass, shear yield system with story-stiffness expressed by Ramberg-Osgood's formula. In this case the vibration equation is as follows;

$$M\ddot{\mathbb{Y}} + (C_E M + C_I S)\dot{\mathbb{Y}} + S\mathbb{Y} = -M\ddot{\mathbb{y}}_0 \tag{2}$$

In this calculation yielding shear of each story is obtained from the assumption of instantaneous yielding at the top and the bottom of the columns at that story, and yielding displacement, is obtained from the yielding shear divided by elastic stiffness. In the Ramberg-Osgood equation, same coefficients α, γ are used as in part I. On the other hand, in frame analysis, stiffness matrix of member is composed as follows;

$$\begin{Bmatrix} M_C^T \\ M_C^B \\ Q_C \\ N \end{Bmatrix} = 2EK_{Cij}\begin{bmatrix} a_c & b_c & c_c & 0 \\ b_c & a_c' & c_c' & 0 \\ c_c & c_c' & d_c & 0 \\ 0 & 0 & 0 & e_c \end{bmatrix}_{ij} * \begin{Bmatrix} \theta_C^T \\ \theta_C^B \\ u \\ \varepsilon \end{Bmatrix}_{ij} \tag{3}$$

$$\begin{Bmatrix} M_G^L \\ M_G^R \\ Q_G \end{Bmatrix} = 2EK_{Gij}\begin{bmatrix} a_G & b_G & c_G \\ b_G & a_G' & c_G' \\ c_G & c_G' & d_G \end{bmatrix}_{ij} * \begin{Bmatrix} \theta_G^L \\ \theta_G^R \\ v \end{Bmatrix}_{ij} \tag{4}$$

$$\{M_P\} = \beta_p[G_P]_{ij}\{\gamma\}_{ij} \tag{5}$$

To take into account the elasto-plastic behavior of members, rigid-plastic springs are inserted at each end of members in this analysis. The stiffness matrix of the member has still same from as eq (3), eq (4) and eq (5) except that the coefficients in eq (3), eq (4) and eq (5) have to be changed. Precise formulae are shown in reference 5. In this calculation, the stiffness of inserted spring K_1 is infinitive in elastic stage and α_1 is assumed to be 0.05 in a yielding condition, namely, K_1 is 5% of elastic bending stiffness of member $2EK$. The vibrational equation, same form as equation (2), is solved step-by-step with linear acceleration method. If the stress in the beam to column connection (panel zone) τ_p exceeds a yielding stress τ_Y, for instance, reduction of shear rigidity of the panel is conducted by changing the value of β_p. In general, the stress of this part is calculated by the following equation.

$$\tau_P = \{-M_C^B + M_C^T + Q_G^R\lambda^R l + Q_G^L\lambda^L l\}/4BDt \tag{6}$$

4. THE EXPERIMENTAL AND THE ANALYTICAL RESULTS

Experiments were tried a few times by changing the intensity of the simulated earthquake, but in this paper, only the case in which plastic hinges were generated at the base of 1-story's columns, is checked by analysis and discussed hereafter. The natural periods obtained just before this test were 0.24 sec. for T_1, 0.09 sec. for T_2 and 0.05 sec. for T_3. In this test run, the record for the N–S component of the EL CENTRO 1940 earthquake was fed into the command center of the actuator except that the intensity was changed. The maximum table acceleration was measured to be 0.944G. The measured responses at each floor level are shown in Fig. 7 together with the calculated responses and their maximum values are listed in Table 3.

The analysis was made in three cases only at the portion of the largest response of the first 10 seconds. The first two cases were based on the frame analysis and the third was shear-model analysis. As the maximum displacement attained at the 1st floor is 0.84 cm and the calculated yield displacement based on shear model type analysis is 0.61 cm, the plastic ratio is about 1.4. The yielding phenomena of the columns were also evident from the strain records observed on their bottom parts. As the panel zones were especially strengthened by steel plates, the strain at this part was observed to be much lower than the yield limit as is anticipated.

Comparing with analytical results of frame analysis, in case of $C_E = 0$ and $C_I = 0.0016$ ($h = 0.02$), acceleration of 3rd floor is 13% smaller ($+1.64G$) than that of experiment ($+1.88G$), but displacement from analysis is $1.2 \sim 1.3$ times larger than experimental data. The calculation shows yielding phenomena only at the bottom of the 1st story columns which explains the experimental results quite well. Similitude of cyclic tendency between measured and calculated results is very good, but the curve obtained from analysis shows drift to plus side.

In the case of $C_E = 1.06$, $C_I = 0$ ($h = 0.02$), although, acceleration amplitude spreads furiously and shows the influence of higher modes, as for displacement good accordance was observed.

In the calculated response of shear type model with Ramberg-Osgood hysteresis with external type of damping ($h = 0.02$), there are tendencies of lower acceleration and larger displacement at each floor. And higher mode wave appears predominantly again in the accelerogram. The displacement prediction, however, was satisfactory.

After all, it may be said that in the calculations, it is irregular as for drift of displacement, but cyclic tendency is almost similar with experiment. As for wave form of accelerogram and displacement, analytical result almost fits in experimental data, however, combination of C_I mainly and C_E a little, seems to be a good adjustment for high mode of vibration.

CONCLUSION

The test results of one-storied frames under impulsive loadings and two-bay three storied frame under simulated earthquake motion are compared with the analysis. In the multi-storied frame analysis, not only usual shear yielding type analysis, but also a general method for inelastic analysis is presented and used. In this analysis, the inelastic behavior is taken into account by inserting conceptual rigid-plastic springs at both ends of members. Shear deformation of the beam-column panel and axial deformation of the column are also included. This method may be quite effective in the analysis of the structures which fail in beam first, and then in the trace of yield mechanism formation under seismic loading. The summary is as follows;

1) The elastic-plastic behavior of one degree of freedom steel structure under dynamic excitation was predicted with use of Ramberg-Osgood hysteresis model or bi-linear hysteresis model and with slight damping value ($h = 1\%$). However, better prediction was obtained by the former within the scope of these experiments.
2) The inelastic behavior of the multi storied steel frame under simulated earthquake motion was predicted satisfactorily by the frame analysis presented herein and with the assumption of slight internal-type damping (2% for the first mode in this test). With the assumption of external-type damping, the displacement response is not so much different from the test results. However, high frequency component appears predominantly in the accelerogram.
3) As the structure was shear type, the shear yielding model with Ramberg-Osgood hysteresis and with same damping value as above was acceptable in the prediction of dynamic response.

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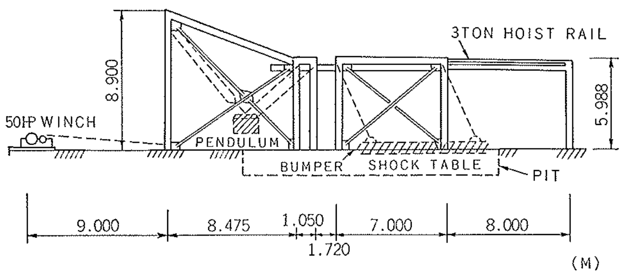

Fig. 1 SHOCK TABLE

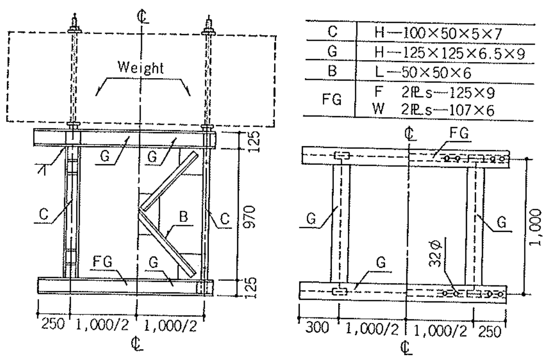

Fig. 2 SI SPECIMEN

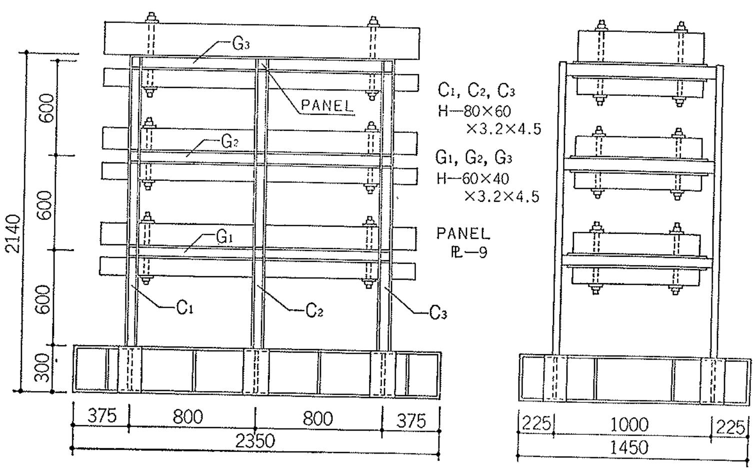

Fig. 3 FRAME SPECIMEN

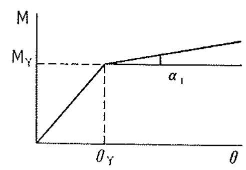

Fig. 4 STIFFNESS REDUCTION AT YIELDING

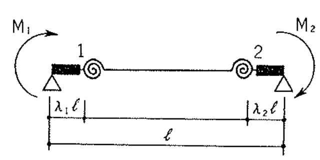

Fig. 5 INSERTED SPRING

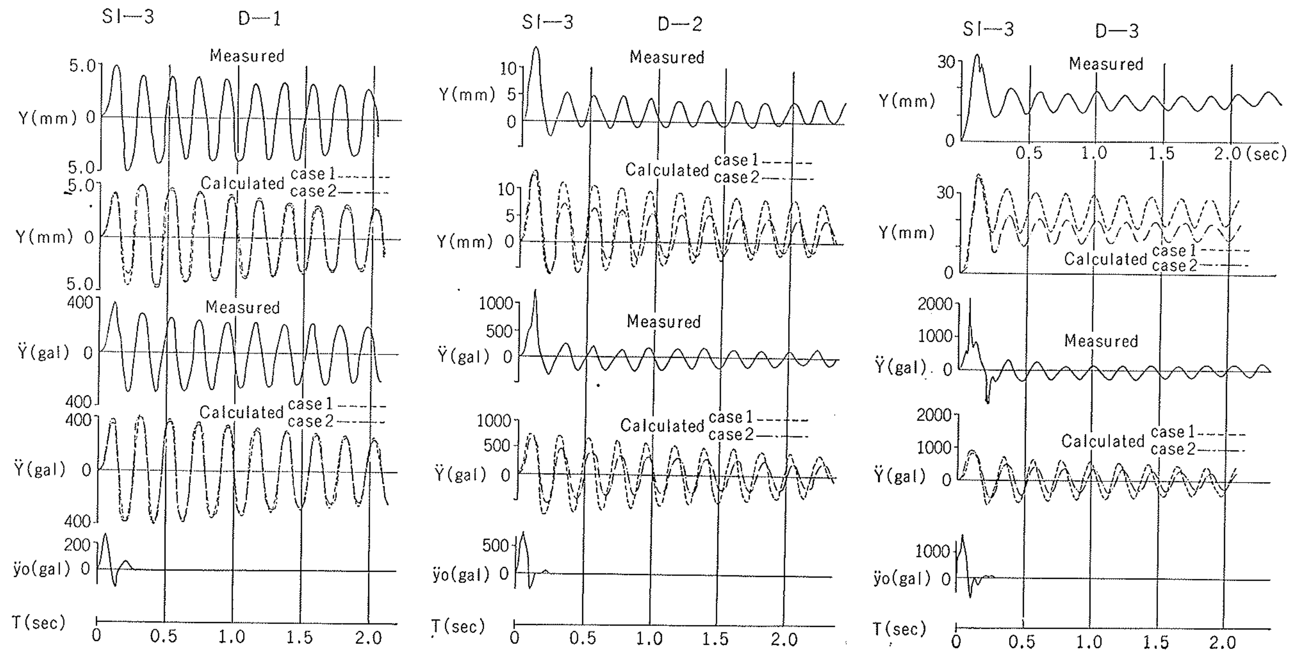

Fig. 6 RESPONSE TO IMPACT WAVES

	σY T/CM²	σMAX T/CM²	E 10^3×T/CM²	ELONGATION (%)
S1	3.09	4.78	2.1	28.0

Table-1 PROPERTY OF SI STEEL

Specimen	Pendulum Height	Measured Natural Period	Measured Damping Constant
	cm	sec	
S1−2	0	.203	—
	10	.207	0.008
	50	.210	.014
	150	.213	.015
S1−3	0	.200	—
	2	.205	.006
	21	.206	.009
	66	.208	.012
S1−4	0	.228	—
	2	.230	.005
	66	.231	.005
	150	.234	.016

Table-2 NATURAL PERIOD AND DAMPING COFFICIENT OF SI TEST RESULT

(ratio)

	(G) Acc. −3 FL	(G) Acc. −2 FL	(G) Acc. −1 FL	(cm) Disp. −3 FL	(cm) Disp. −2 FL	(cm) Disp. −1 FL
EXPERIMENT	+1.88 −1.8	+1.24 −1.12	+1.01 −0.88	+1.78 −2.0	+1.28 −1.40	+0.76 −0.84
FRAME h=0.02 C I	+1.64(0.83) −1.6 (0.89)	+1.26(1.02) −1.14(1.02)	+1.0 (1.0) −1.42(1.61)	+2.08(1.17) −2.4 (1.2)	+1.66(1.30) −1.84(1.31)	+1.0 (1.32) −1.0 (1.19)
FRAME h=0.02 C E	+1.6 (0.85) −0.92(1.07)	+2.16(1.74) −2.0 (1.78)	+2.28(2.28) −2.24(2.55)	+1.92(1.08) −2.36(1.18)	+1.44(1.13) −1.72(1.23)	+0.84(1.11) −1.10(1.31)
Ramberg−Osgood h=0.02 C E	+1.44(0.77) −1.76(0.98)	+1.26(1.02) −1.44(1.29)	+1.08(1.08) −0.92(1.05)	+2.25(1.42) −2.24(1.12)	+1.88(1.47) −1.72(1.23)	+0.92(1.21) −0.84(1.0)

Table-3 MAXIMUM VALUE OF MEASURED AND CALCULATED RESPONSE OF FRAME TEST

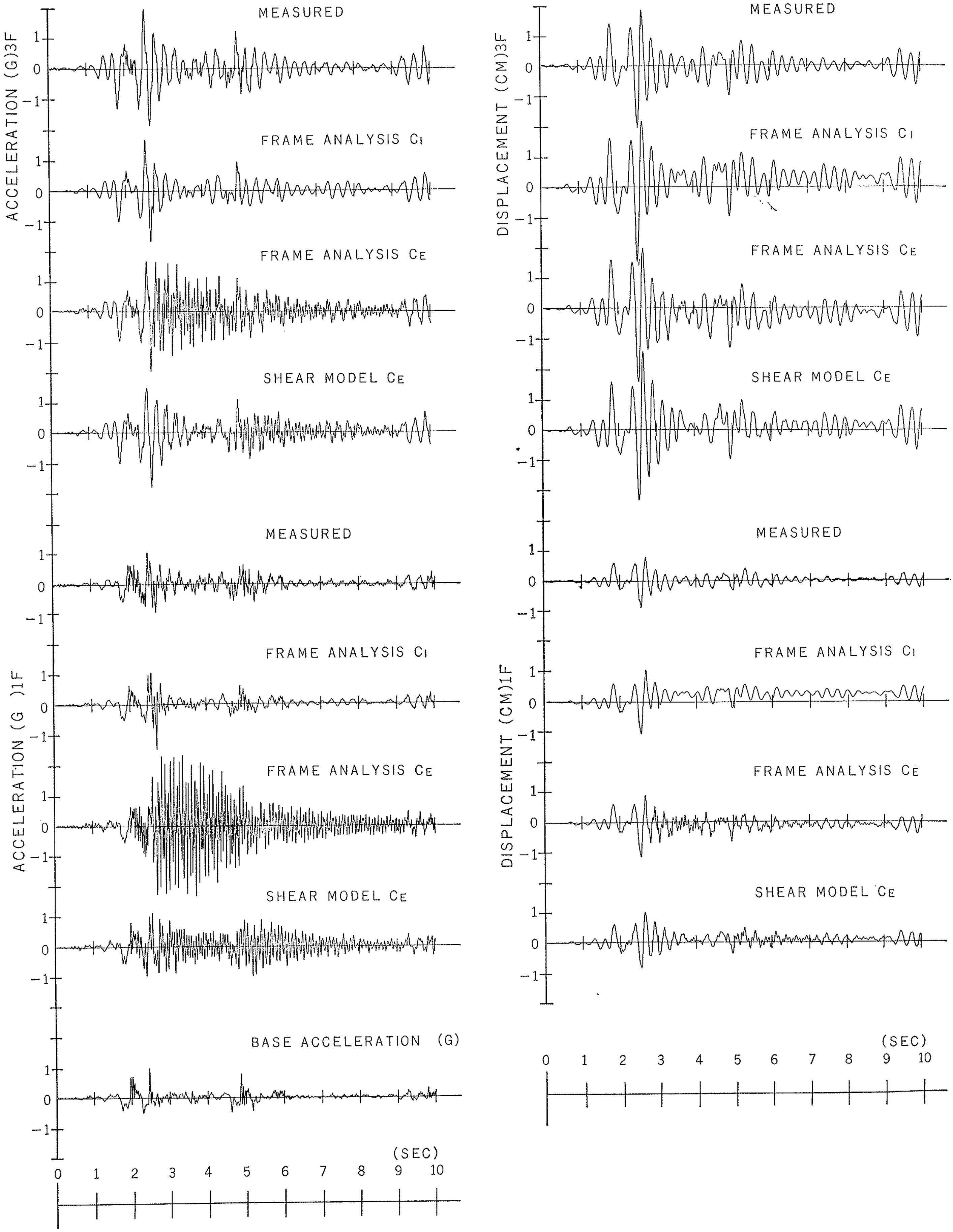

Fig. 7 RESPONSE TO SIMULATED EARTHQUAKE