BRIEF NOTE

A SIMULATION OF EARTHQUAKE RESPONSE OF STEEL FRAMES

by

Koichi TAKANASHI[I], Kuniaki UDAGAWA[II], Hisashi TANAKA[III]

Description of Procedure

This paper describes a part of simulation results of the earthquake response of steel structures as a preliminary report. As reported precisely in the previous paper[1], the authors have already developed a system for the dynamic analysis using a computer-and-computer-controlled dynamic testing machine which are connected on line, where exact analyses are possible depending on the real restoring forces of structures obtained by the experiments on structures or structural elements. The dynamic analyses of steel frames have been carried out so far by this system. A part of frames analyzed are summarized in Table 1. All frames are one bay-one story with strong columns and weak beams. The columns are assumed entirely elastic throughout response, but beams respond elastically and plastically. Then, beam tests are needed to obtain the exact restoring characteristics of the frames. In Fig. 1, the frame and the test beam are shown schematically. The end moment vs. end rotation characteristics of the inelastic beam at a time can be completed by measuring continuously the bending moment and the deflection at the center of the beam specimen below.

Dimensions and Natural Periods of the Frames

The beams of all frames have the H-sections of H-200x100x5.5x8, the length of which is designed 130 cm to be adapted to AIJ recommendations of Plastic Design of Steel Structures (in the recommendations, $\ell h/A_f$ shall be less than 375 for SS·41 Steel). The test specimens were also made of the same sections. The columns, the height H of which is 120 cm, are assumed the H-sections of H-175x175x7.5x11. The plastic strength of the columns is more than that of the beams. These frame models are scaled 1/2.5 of the prototype.

The natural periods of the frames are determined 0.4 sec and 0.6 sec a priori. Therefore, the masses m, which are considered to be concentrated at the roof level, must be adjusted to satisfy the relation between the natural period and the stiffness of each frame. The axial forces of the columns due to the masses were

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I) Associate Professor,
II) Research Associate,
III) Professor, Institute of Industrial Science, University of Tokyo, Tokyo, Japan

1) Takanashi K. et al "Non-linear Earthquake Response Analysis of Structures by a Computer-Actuator On-Line System" Bulletin of ERS No. 8 Dec. 1974

calculated and taken into account in the response calculation as P-Δ effect.

Ground Accelerations

The two kinds of ground motions were adopted in the analyses; the sinusoidal motion with the period of 0.5 sec and the earthquake motion recorded at El Centro in 1940. The maximum values of the accelerations were proportioned by the coefficients of the yield accelerations α_Y, as shown in Table 1. The yield acceleration α_Y is defined as the ratio Q_Y/m, where Q_Y is the story shear force at the commencement of yielding in the beam. The duration times of both ground motions are 10 sec and followed by the free vibration of 2 sec.

Results of Analysis

Two results are picked up to show the different types of responses. The results are shown in Figs. 2 and 3. In each figure the end moment M/M_Y vs. the end rotation θ/θ_Y of the beam, and the story shear, Q/Q_Y vs. the displacement response X/X_Y relationships are shown in (a) and (b), respectively. The time histories of the ground acceleration $\ddot{X}_0$, the end rotation θ/θ_Y, the end moment M/M_Y, the displacement response X/X_Y, the story shear Q/Q_Y and the stiffness of the frame K/K_E are shown in (c), where θ_Y and X_Y are the rotation and the displacement at the commencement of yielding, respectively, and K_E is the elastic stiffness.

The results of Frame 2 with the natural period of 0.4 sec in Fig. 2 show the typical collapse of steel frames by the ground motion. The restoring force Q decreases due to the plastic lateral buckling of the beam. The displacement X shifts to the one side. On the other hand, Frame 4 with the period of 0.6 sec did not fail. The displacement response stayed within the limited extent. It should be noted that the natural period of Frame 2 is less than the period of the sinusoidal ground motion, but the period of Frame 4 is longer than that of the ground motion. An example of the response to the earthquake motion is Frame 5 and is shown in Fig. 3. The frame did not fail in this case.

Adaptability of Analytical Models

Non-linear hysteresis loops such as bi-linear, tri-linear, Ramberg-Osgood type function etc. have been widely adopted as analytical models of restoring characteristics of frames for the earthquake response analysis. However, the examination of their adaptability have been quite few. In Fig. 5, the calculation results of the maximum end rotation subjected to the sinusoidal ground motion are plotted, and in Fig. 6 the maximum values of the end rotation of the beam subjected to the El Centro earthquake are plotted. These calculations were done assuming the hysteresis loops of the beams to be Ramberg-Osgood type (the coefficients of the function are determined to best fit the result of the cyclic test with a constant amplitude) and bi-linear function (the second slope is 0.2 times the first). The open marks in these figures show the maximum values obtained by authors' system of the computer-and-computer-controlled testing machine. As recognized by the figures, the analytical model of Ramberg-Osgood type function can predict well the maximum value of response except the cases where the response values are quite large and/or shifted to the either side. In other words the analysis by the analytical models can not predict entirely the behavior near collapse.

Ductility or Rotation Capacity

It is widely recognized that structures should have enough ductility and the members should show enough rotation capacity. And also it is a question yet what is the enough ductility. The rotation capacities of beams subjected to the monotonic loading and the cyclic loading have been obtained by the authors as shown in Fig. 4. The maximum rotation response of the beams under ground exitations are shown in Figs. 5 and 6 with the above mentioned rotation capacities of the monotonic and cyclic tests shown with chain lines. These three results are comparatively examined to discuss the permissible rotation of the beam.

The curve for the monotonic loadings represents the experimental results of the end rotations at the maximum end moments, and the curve for the cyclic loadings represents the amplitudes of the end rotations, within which no reduction of end moment occurs and therefore the stable hysteresis loops can be obtained.

The response of Frame 4 ($T_0/T = 1.2$) in Fig. 5 stays within the cyclic stabilty limit. It is considered quite safe because there is no loss of the strength. But the response of Frame 3 ($T_0/T = 0.8$) is unstable and going far beyond the stability limit of monotonic tests. The beam in this case was so damaged that it cannot be expected to have serviceability.

Contrary to the cases of sinusoidal excitations mentioned above, the responses of Frame 5 and Frame 6 to the earthquake ground motion show the different features. It is hardly said that Frame 6 ($T_0 = 0.4$, $1.5\,\alpha_Y$) failed down, although the maximum response is above the stability limit of monotonic loading. The reason is that only a few peaks of the response go beyond the cyclic stability limit and then the loss of strength was not so serious as to cause the frame failure. Frame 5 ($T_0 = 0.5$, $1.0\,\alpha_Y$) is of course quite safe.

Generally speaking, the design criterion that the maximum response to an earthquake motion must be under the cyclic stability limit, is too conservative. But the establishment of more economical design criterion requires the more precise research, because the behavior of a frame in the region above the cyclic stability limit depends on how many times and how far beyond the limit the peaks of response excurse.

Table 1. Summary of Frames Analyzed

Frame No.	Beam		Column	Stiffness ratio	Natural period	Axial load	Ground motion
	Specimen No.	l(cm) lh/A_f	H(cm) H/r_x	$\frac{I_b/l}{I_c/H}$	T_0(sec)	N/N_y	Type Intensity
1	EX-1	130 325	120 16	0.467	0.4	0.034	sin 1.5 α_Y
2	EX-2	130 325	120 16	0.461	0.4	0.034	sin 1.2 α_Y
3	DG-130-12	130 325	120 16	0.458	0.4	0.034	sin 1.0 α_Y
4	DG-130-13	130 325	120 16	0.436	0.6	0.074	sin 1.0 α_Y
5	DG-130-14	130 325	120 16	0.444	0.4	0.033	El Centro 1.0 α_Y
6	DG-130-15	130 325	120 16	0.450	0.4	0.034	El Centro 1.5 α_Y

1) Beam H-200×100×5.5×8, Z_p=209 cm^3, A_f=8.0 cm^2, h=20 cm, $lh/A_f \leq 375$ by AIJ
2) Column H-175×175×7.5×11, Z_p=369 cm^3, A=51.21 cm^2, N_y=3.0×51.21=153.63 t
3) Sinusoidal ground motion, Period, T=0.5 sec
4) El Centro NS, 1940

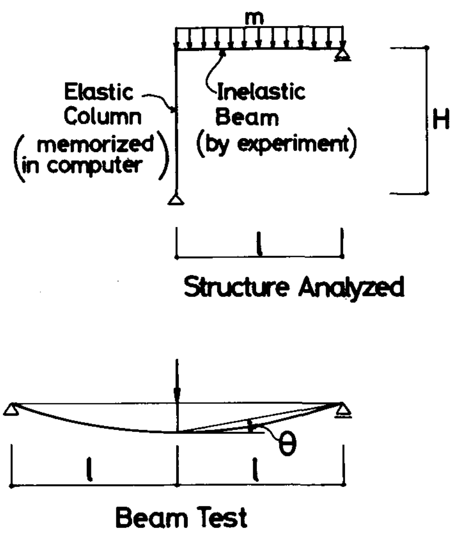

Fig. 1 Frame and Test Beam

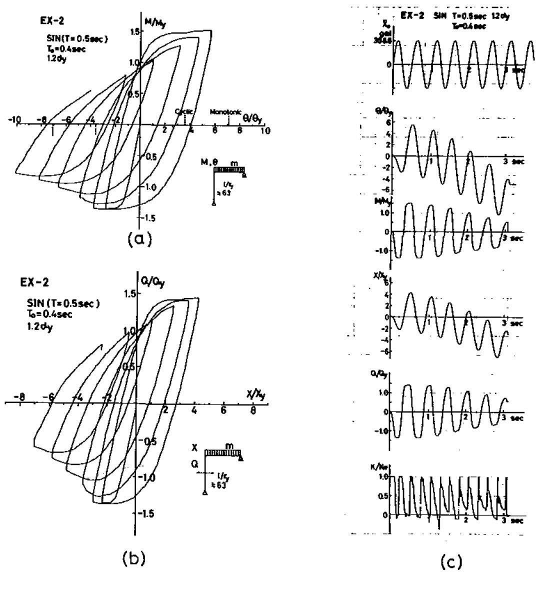

Fig. 2 Results of Frame 2

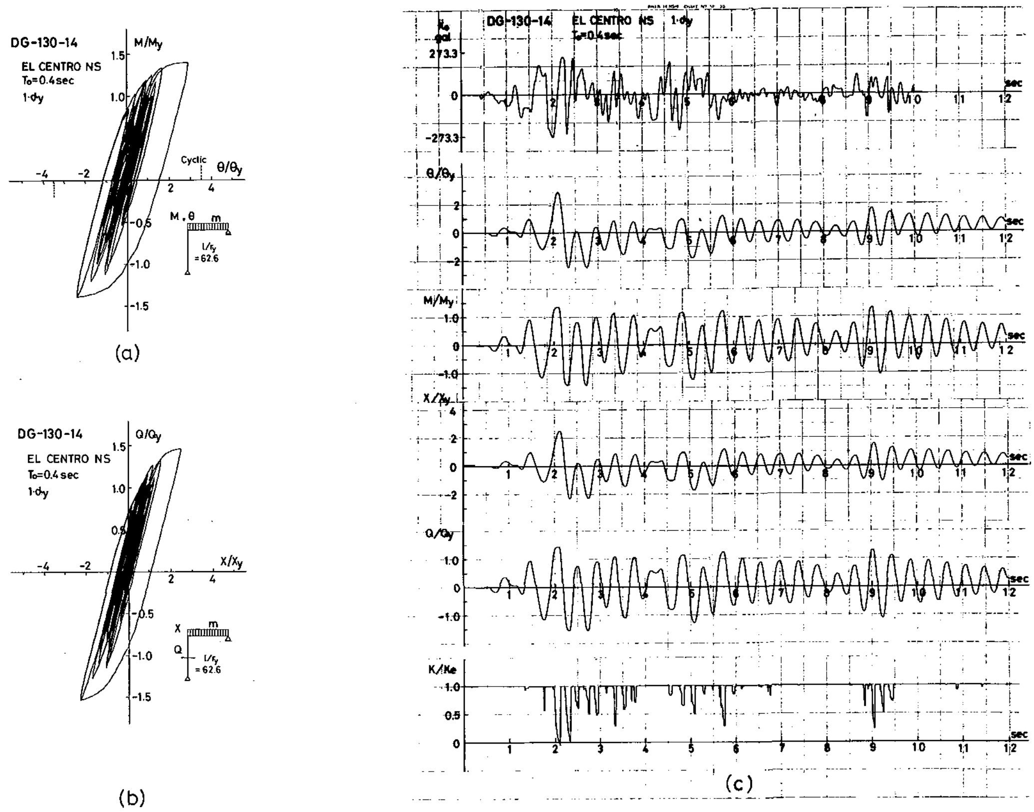

Fig. 3 Results of Frame 5

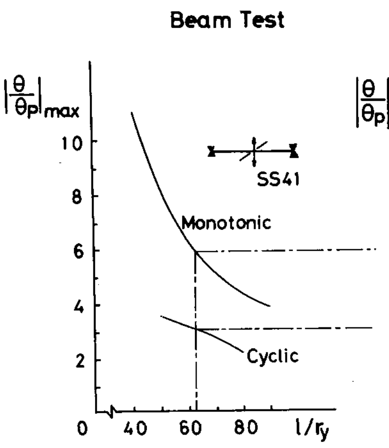

Fig. 4 Rotation Capacities of Beams

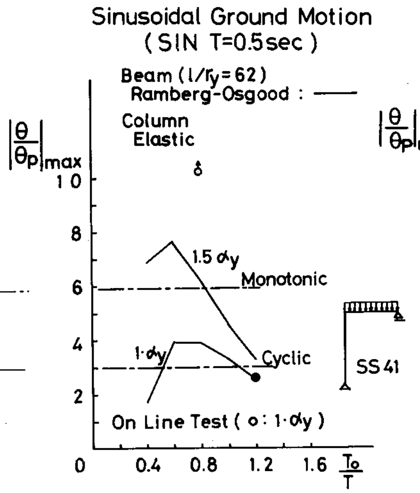

Fig. 5 Sinusoidal Response

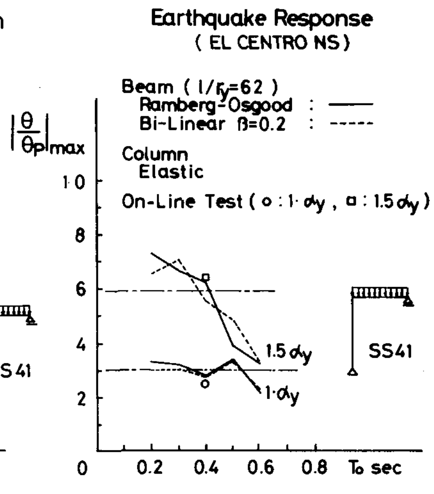

Fig. 6. Earthquake response