A SIMULATION OF EARTHQUAKE RESPONSE OF STEEL BUILDINGS

by

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SYNOPSIS

This paper presents a part of ongoing research on seismic design criterion for steel buildings and describes simulation results of the earthquake response of steel structures with inelastic beams. The response analysis was made by a "Computer-Actuator On-Line System" developed by the authors (1). In order to maintain the stability of structures under a strong earthquake, the permissible magnitude and number of excursions of plastic response deformations of beams were discussed in comparison with the rotation capacities obtained by the monotonic and the cyclic reversed loading tests of beams (2),(3). The earthquake response of the analytical models of the same structures as those of on-line tests were also calculated assuming the hysteresis loops of beams to be Ramberg-Osgood type and bi-linear functions, and the adaptability of analytical model analyses was investigated.

INTRODUCTION

From the experimental research of steel beams under monotonic and cyclic reversed loadings, the plastic behaviors of beams were well examined and the rotation capacities (deformations at the stability limit) under both loading conditions were determined (2),(3). Nevertheless, the plastic behavior of steel beams due to earthquake loadings is not clarified yet. In the case of dynamic response for earthquake loadings the instability of the restoring force characteristics are anticipated at the large response displacements since such an instability was occurred beyond the stability limit in the cyclic reverse tests with constant amplitudes. Therefore, it is necessary to carry out the analysis by the computer-actuator on-line system and to establish the criterion of the seismic design taking account of the critical values of ductility and the deterioration of strength.

COMPUTER-ACTUATOR ON-LINE SYSTEM

As reported precisely in the previous paper (1), the authors have already developed the computer-actuator on-line system for the earthquake response analysis. By this system, the precise analysis is possible even for a frame whose restoring force is deteriorated at large displacements, without assuming any analytical model of complicated hysteresis loops. In solving the equation of motion, the step-wise linear calculation is adopted, evaluating the instantaneous restoring forces in each step by measuring continuously the deformations of the specimen at the test which is running simultaneously with the calculation.

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ANALYSED FRAMES AND TEST BEAMS

The analysed frames are portal frames with pinned feet and their model frames are scaled 1/2.5 of the prototypes. By the symmetry only the assemblages cut out of the frames, as shown in Fig.1(a), are treated. In the following, the columns are considered entirely elastic throughout the analysis, but the beams behave plastically during severe ground motions. Then, the beams must be loaded in the test machine in order to obtain the exact restoring characteristics. The beam tests were carried out by the simple beam in Fig.1(b). The characteristics of beams could be known by measuring the deflections at the center of the beams.

The beams of all frames have sections of H-200×100×5.5×8 and the length of 130 cm, to be adapted to AIJ Recommendations of Plastic Design of Steel Structures (4) (in the Recommendations, $\ell h/A$ 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 120 cm, are assumed section of H-175×175×7.5×11 and memorized in computer. The plastic strength of the columns is more than that of the beams.

The natural periods T_o 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, and the axial forces of the columns were taken into account in the response calculation.

GROUND ACCELERATION

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 of both ground motions is 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 as 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 story displacement response X/X_Y relationships are shown in (a) and (b), respectively. The time histories of the ground accelerations $\ddot{X}_o$, 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 F-2 with the natural period of 0.4 sec in Fig.2 show the typical collapse of steel frames by the sinusoidal ground motion. The restoring force Q decreases due to the plastic lateral buckling of the beam. The displacement X shifts to one side. As an example of the response to the earthquake motion, the results of Frame F-5 are shown in Fig.3, and this frame did not fail.

ADAPTABILITY OF ANALITICAL 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 for the earthquake response analysis. However, the examination of their adaptability have been quite few. In Figs.5 and 6 the calculation results of the maximum end rotation of the beam are plotted, subjected to the sinusoidal ground motion and to the El Centro earthquake, respectively. These were calculated assuming the hysteresis loops Ramberg-Osgood type (the coefficients of the function are determined as to fit best the result of the cyclic test with a constant amplitude) and bi-linear function (the second slope is 0.2 times of the first, β=0.2). The open marks in these figures show the maximum values obtained by the author's on-line system. As recognized by the figures, the analytical model of Ramberg-Osgood function can predict well the maximum values of response except the case where the response values are quite large and/or shift to 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 enough rotation capacity. But it is still a question what is the enough ductility. The rotation capacities of beams subjected to the monotonic and the cyclic loadings have been obtained by the authors as shown in Fig.4. 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. They are called here the monotonic stability limit and the cyclic stability limit, respectively. The maximum rotation response of the beams under ground exitations are shown in Figs.5 and 6 and compared with the above mentioned rotation capacities, which are shown by the chain lines of the monotonic and the cyclic tests, in order to discuss the permissible rotation of the beam.

The response of Frame F-4 (T_o/T=1.2) in Fig.5 remains within the cyclic stability limit. It is considered quite safe because there is no loss of the strength. But the response of Frames F-1,F-2 and F-3 (T_o/T=0.8) are unstable and going far beyond the monotonic stability limit. The beam in this case was extremely damaged due to severe lateral buckling. It should be noted that the natural period of Frame F-2 is less than the period of the sinusoidal ground motion, but the period of Frame F-4 is longer than that of the ground motion.

Contrary to the cases of sinusoidal excitations mentioned above, the responses of Frame F-5 and Frame F-6 to the earthquake ground motion show the different features. In the case of Frame F-6 (T_o=0.4, 1.5σ_Y) the maximum response is above the monotonic stability limit, but it did not collapse. The reason is that only a few peaks of the response go beyond the cyclic stability limit and then the deterioration of strength was not so serious as to cause the frame failure. Frame F-5 (T_o=0.4, 1$\cdot\sigma_Y$) is of course quite safe, since the maximum rotation is below the cyclic stability limit.

CONCLUSION

The earthquake response analyses of steel structures with inelastic beams, subjected to the sinusoidal ground motions and the earthquake motion recorded at El Centro in 1940, are carried out by the computer-actuator on-line system and the plastic response behaviors of beams during earthquakes are investigated. Following considerations are presented. If the maximum response displacements of beams are within the cyclic stability limit, the beams have the sufficient strength and ductility, not depending on the number of deformation peaks. However, when the beams are repeatedly deformed beyond the monotonic stability limit, the strength and the ductility of beams reduce remarkably and the frames fail down, because the cumulative lateral buckling displacement becomes so large. On the other hand, in case that the response rotations of beams remain between the cyclic and the monotonic stability limit, some frames fail while others don't. This fact depends on the number of peaks and the magnitude of response displacements of beams above the cyclic stability limit.

In general, it seems too conservative such design criterion that the maximum response to an earthquake motion should be under the cyclic stability limit. But the establishment of more economical criterion requires to clarify the relation between the strength deterioration and the peak frequency of deformations in the region above the cyclic stability limit, because the behavior of a frame depends on how many and how large excursions repeat in the response beyond the cyclic stability limit.

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TABLE 1 SUMMARY OF FRAMES ANALYZED

Frame	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
F-1	EX-1	130 325	120 16	0.467	0.4	0.034	sin 1.5 α_Y
F-2	EX-2	130 325	120 16	0.461	0.4	0.034	sin 1.2 α_Y
F-3	DG-130-12	130 325	120 16	0.458	0.4	0.034	sin 1.0 α_Y
F-4	DG-130-13	130 325	120 16	0.436	0.6	0.074	sin 1.0 α_Y
F-5	DG-130-14	130 325	120 16	0.444	0.4	0.033	El Centro 1.0 α_Y
F-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

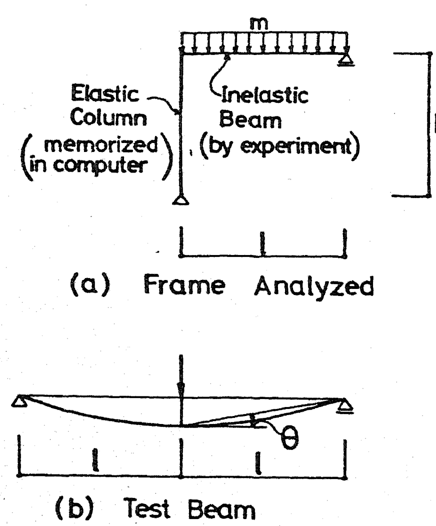

Fig.1 Frame Analyzed and Test Beam

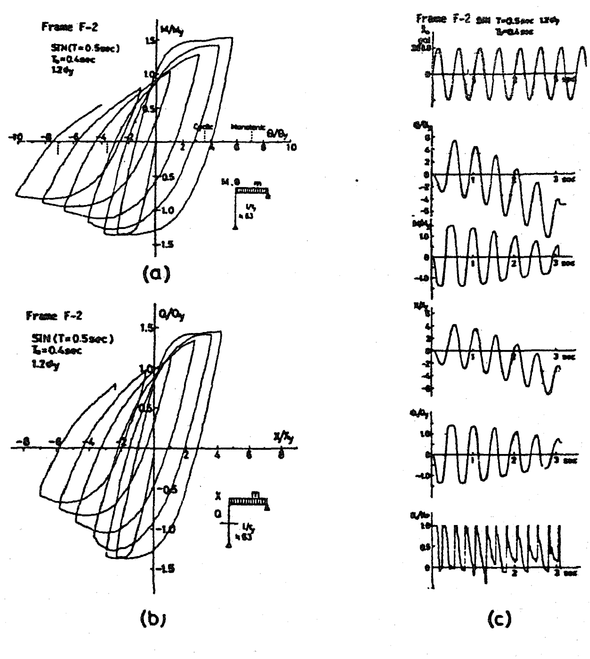

Fig.2 Results of Frame F-2 (Sinusoidal Ground Motion)

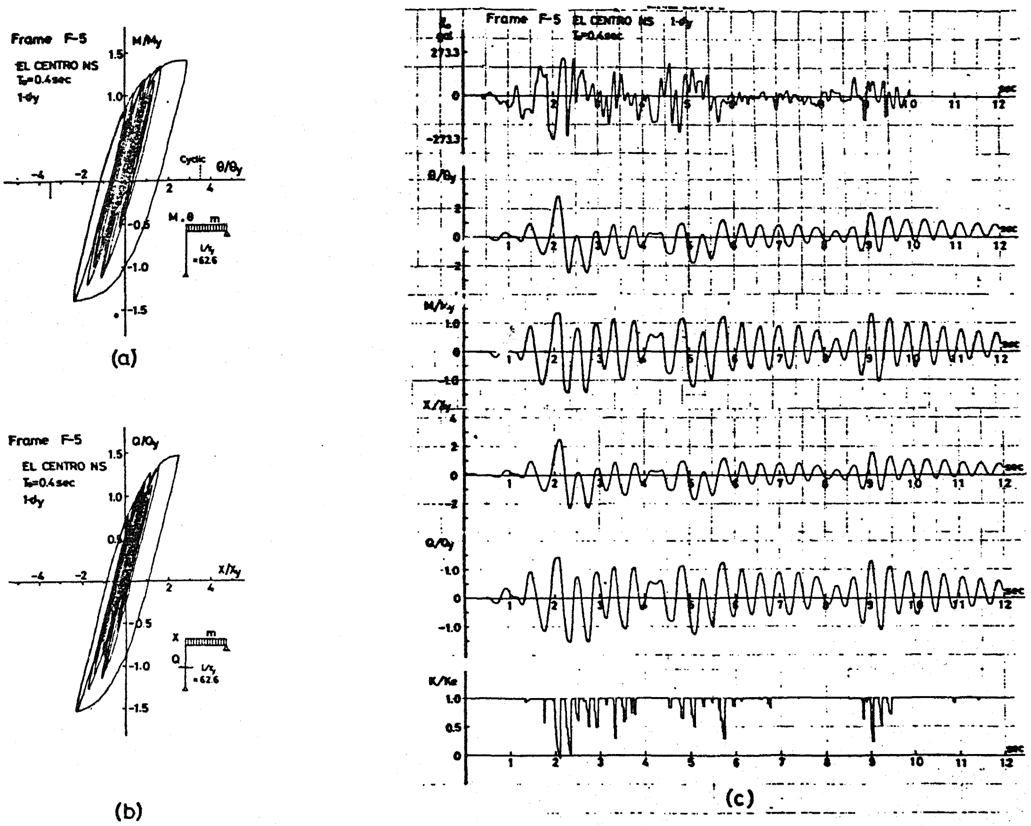

Fig.3 Results of Frame F-5 (Earthquake Motion)

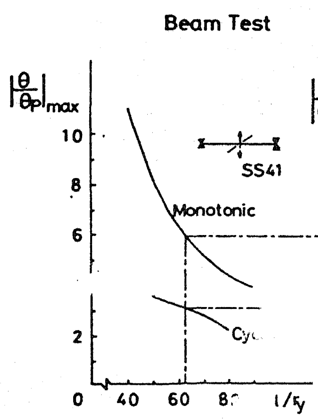

Fig.4 Rotation Capacities of Steel Beams

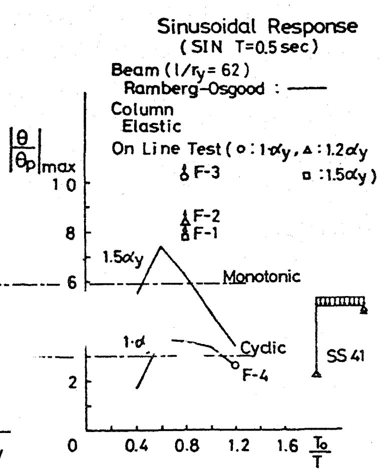

Fig.5 Sinusoidal Response

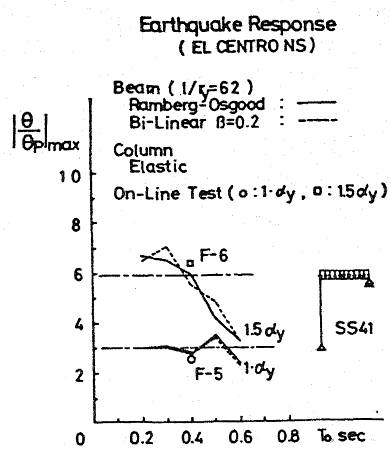

Fig.6 Earthquake Response

DISCUSSION

H. Aoyama (Japan)

Since Professor Takanashi and his group is the first in the world to be quite successful in structural dynamic testing using actuator computer on line system, The discussor would like to have a few words on the keys to the success of this type of testing. In particular, what loading speed was selected and what smoothing technique is used in A-D conversion ?

Author's Closure

Authors appreciate Prof. H. Aoyama's valuable comments on our works and his keen insight into the keypoints of our experimental techniques. The satisfactory results depend on the following two fundamentals in the dynamic analysis by our computer-actuator on-line system;

(1) Accurate measurements of deformations and loads,
(2) Adoption of an adequate numerical intergration technique.

To attain the first item, a pause must be needed between the time intervals in the step by step calculation, so that the controlled displacement of actuator acting on a specimen can settle at its specified position and a lot of sumpling data can be measured to make an average value.

To attain the second item, we tried the three kinds of integration methods. Our conclusion is that the best method must be selected among them for a specified test. For example, the central difference method is suitable for the dynamic analysis of a structure with the complicated restoring characteristics. Moreover, it brings another advantage that the direct use of the value of restoring force, which eliminates the influence of errors in the measurements at the tests on the calculation of results.