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Inelastic response of steel structure subjected to two horizontal ground motions

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ABSTRACT: The objective is primarily concerned to clarify the dynamic response and the resultant aseismic safety criteria of steel moment resisting spacial structures, subjected to two horizontal components of destructive earthquakes, based on a shaking table experiment. The experimental results have shown that the responses of the weak beam type structures can be treated separately along the principal axes. Whereas, in the case of weak column type structures, the interaction effect can be significantly observed, especially for the structures in which P-Δ effect should be taken into consideration.

1 INTRODUCTION

During past two decades, dynamic response and failure mechanism of steel moment resisting building structures in the event of earthquakes, have been widely investigated, both experimentally and analytically. The evidence deduced from the findings has been applied in order to establish aseismic safety criteria of the structures.

However, since almost all past investigations may be limited in the case when the structures would be subjected to the single components among horizontal earthquakes, a question arises whether the criteria derived from the past investigations, could be directly applied or not, in assessing the structural safety under biaxial ground motions. It can be anticipated that the dynamic behaviors of the spacial structures become more complex because of the difference of the failure mechanisms of the individual planer frames consisting of the overall spacial structures, anisotropy of the column members, the floor diaphragm effect, and so on.

E.Rosenblueth and H.Contreras have shown that the combining effect on the structural response of various ground motion components can be approximated by a simplified formula equated with the sum of the maximum responses under the individual component i, multiplied by the magnification factor αi.

This paper primarily concerns with the interaction effect on the ductility demand of the typical steel structures subjected to biaxial ground motions, based on a shaking table experiment.

2 EXPERIMENTAL INVESTIGATION

Single one story spacial structures consisted of identical four planer frames were used to vibrate on a shaking table, as shown in Figure 1. The columns and the beams were made of rectangular hollow sections, and the each column was connected by welding to a rigid steel lump at the top, so as to avoid the panel deformation effect. The lump was directly connected to the beam ends at the orthogonal two directions. Special universally rotating crevices were attached at the column bases so that the bases can rotate freely around the two principal axes of the structures.

Any other roof members except the perimeter roof beams were not arranged in order to avoid the stiffening effect on the strength of the spacial structures. Only the weight was mounted at each corner of the roof plane so as to generate the inertia force , as illustrated in Figure 1.

The test structures consisted of two different types in its failure mechanisms. Namely, one is so-called weak beam type structure and another is weak column type structure, named as B-Type and C-Type, respectively.

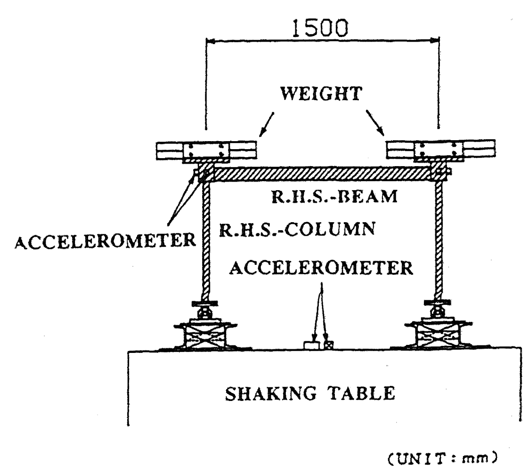

Figure 1. Test set-up

The ground acceleration records obtained from Mexico Earthquake in 1985, were used to excite the table as the input motions. It has been reported that these records were measured on the grounds consisted of comparatively soft soils and many buildings in the sites were severely damaged because of the magnitude and the length of duration of the earthquake. It can be anticipated that the weak beam type structures may suffer so called low cycle fatigue damage because of the long duration of the earthquake, and the weak column type structures also may be lead to incremental collapse or single excursion failure due to the nature of the close predominant periods in two horizontal components of the earthquake.

The fundamental properties of the original acceleration records are tabulated in Table 1. The durations of the records were scaled down to coincide the natural periods of the test structures with those of the corresponding real structures. The scaling factors are shown in Table 2, together with the values of the fundamental properties of the test structures.

Each type of the structure mounted on the table was accelerated along the single direction parallel to the planer frame, or two principal directions of the structure, by applying the modified acceleration records to the system. A pair of accelerometers were set along the two orthogonal directions on the shaking table to measure the input accelerations to the overall vibrating system composed of the test structure and the table, and absolute acceleration responses were detected by a pair of accelerometers set up at each column top. The local strains along the longitudinal directions at the critical points near the beam-to-column connections were also measured by the plastic strain gauges.

3 EXPERIMENTAL RESULT AND DISCUSSION

3.1 Data generating procedure

After the experiment, the recorded acceleration data were numerically filtered to generate noise free data. Then, the time domain outputs such as the relative velocities and the story drifts, were generated through numerical integration and filtering. The numerical processing technique has already been reported by the authers.

3.2 Response of B-Type structure

The results of dynamic responses in weak beam type structures are tabulated in Table 3. From the relative displacement trajectories due to single components of excitations, it may be seen that the structure behave only along the direction of the input motion, even after the yielding. The vibration response orthogonal to the direction of the input motion can be observed only after the crack propagation at the beam ends. This phenomenon indicates that the responses as the whole spacial frames are not primarily influenced due to the absence of the roof diaphragms, if the members are arranged

Table 1. Ground acceleration records of Mexico Earthquake in 1985

name	duration (sec)	max. value (gal)	dominant frequency (Hz)
M6T	58.04	166.95	0.49
M6L	54.14	97.85	0.48
M11T	61.26	111.49	0.28
M11L	60.33	117.49	0.48

Table 2. Properties of test structures

name	αy (gal)	Dy (mm)	T (sec)	SF	acc.record
B1	720	16	0.29	0.2	M11L
B2	720	16	0.29	0.2	M11L,T
B3	341	25	0.54	0.25	M11T
B4	341	25	0.54	0.25	M11L,T
C1	459	30	0.51	0.25	M11L,T
C2	191	8	0.40	0.2	M6T
C3	191	8	0.40	0.2	M6T,L

αy : absolute acceleration at the first yielding
Dy : relative displacement at the first yielding
T : natural period of vibration
SF : scaling factor of the original duration

Table 3. Maximum response of B-Type strucutre

structure's name	B3	B4	
input acceleration's name	M11T	M11T	M11L
direction	X	X	Y
input acceleration (gal)	287	288	317
time of occurrence (sec)	9.9	9.9	4.8
response acceleration (gal)	394	462	420
time of occurrence (sec)	9.8	9.8	7.5
relative displacement (mm)	35	35	43
time of occurrence (sec)	9.9	9.9	10.7
ductility factor	1.38	1.40	1.72
strain (%)	1.04	0.88	1.41

Table 4. Maximum response of C-Type strucutre

structure's name	C2	C3	
input acceleration's name	M6T	M6T	M6L
direction	X	X	Y
input acceleration (gal)	41	58	59
time of occurrence (sec)	8.7	8.4	8.7
response acceleration (gal)	170	193	222
time of occurrence (sec)	8.9	8.9	8.9
relative displacement (mm)	7	6	8
time of occurrence (sec)	8.7	8.5	8.5
ductility factor	0.91	0.78	0.97
strain (%)	0.19	0.19	0.26

symmetrically and the material strengths of the individual members are few scattered.

Under bidirectional horizontal ground motions, the response displacements at the column tops show the quite similar trajectories even in the moderate plastic regions, as exemplified in Figure 2. In the figure, dashed lines show the yield displacement of the structure. Such phenomenon indicates that the structures can behave in the bodies, unless the perimeter beams deteriorate due to the crack initiation and propagation under repetitive cyclic loadings. For B-Type structures, since the columns behave within its elastic range, and only the beams can sustain the inelastic distortions and dissipate energy, the overall responses can be separated into independent responses correspondent to the horizontal components of the ground motions.

From the result, it has been demonstrated that aseismic safety limit of the weak beam type structures can be settled by low cycle fatigue failure at the critical sections of the individual beams, if the beams would efficiently stiffen against both torsional and local buckling.

3.3 Response of C-Type structure

In C-Type structures, biaxial interaction effect significantly differ in degree, whether the structures respond within elastic or slightly plastic range, or far from the elastic limit.

Table 4 shows the result of the maximum response under the single component or two components of ground motions, where the structures behave slightly beyond the elastic limit. From the table, the effect due to two directional motions can be insignificantly observed.

The responses of the structures with difference in failure mechanism under two components of horizontal excitations were compared, and the results are tabulated in Table 5. From the table, it can be seen that the difference of the failure mechanism scarcely affect the dynamic response, if the ductility response is within the range of about 2.0. Figures 3 and 4 show the trajectories of the relative displacements in the roof planes. In Figure 4, it can be observed that the trajectories of Column 1 and 2 are somewhat different from the others. It is because the beam connected to these columns has deteriorated due to the crack propagation at the critical section.

However, in the case of C2 and C3 structures, where the excessive story drift may result in the overall collapse of the structure due to so called P-Δ effect, the interaction effect can be clearly observed, as shown in Table 6. The dominant frequencies of the acceleration records, used as the input motions, are very close as referred in Table 1. It has been recognized that the characteristic of the input motions let the structure vibrate only along the particular direction, and lead to collapse. The trajectories of the displacements at the column tops are shown in Figure 5. The irregularity of the trajectories at the final stage, indicates that the structure is suspended through wire cables by a rigid frame installed in order to avoid the demolition of the equipment. The result has been demonstrated that the collapse under two directional

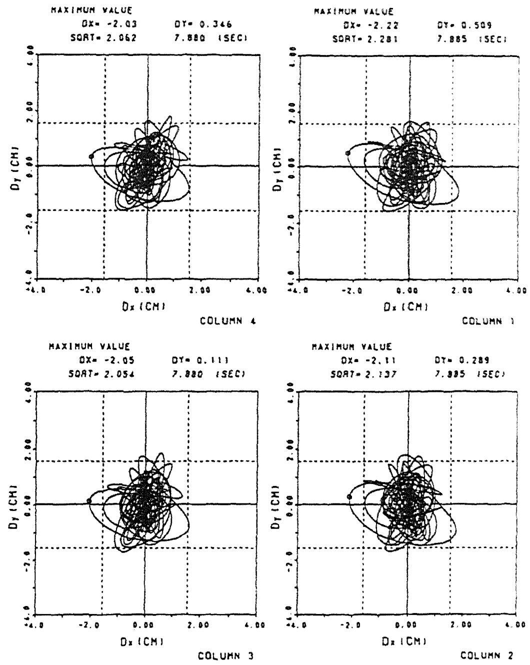

Figure 2. Displacement trajectories of B2 structure

Table 5. Comparison of maximum response between B and C-Type strucutres

structure's name	B4		C1	
input acceleration's name	M11T	M11L	M11T	M11L
direction	X	Y	X	Y
input acceleration (gal)	288	317	211	233
time of occurrence (sec)	9.9	4.8	10.0	9.1
response acceleration (gal)	462	420	407	541
time of occurrence (sec)	9.8	7.5	10.0	9.2
relative displacement (mm)	35	43	31	44
time of occurrence (sec)	9.9	10.7	9.7	9.3
ductility factor	1.40	1.72	1.04	1.46
strain (%)	0.88	1.41	0.13	0.17

Table 6. Maximum responses of C2 and C3 strucutres

structure's name	C2	C3	
input acceleration's name	M6T	M6T	M6L
direction	X	X	Y
input acceleration (gal)	294 [217]	272[189]	360[287]
time of occurrence (sec)	8.4[7.7]	8.7[5.0]	8.4[5.0]
response acceleration (gal)	249	80	226
time of occurrence (sec)	7.9	6.5	6.5
r.m.s. of response acceleration (gal)	249	240	

numerals in [ ] indicate the values measured until the structures has just collapsed.

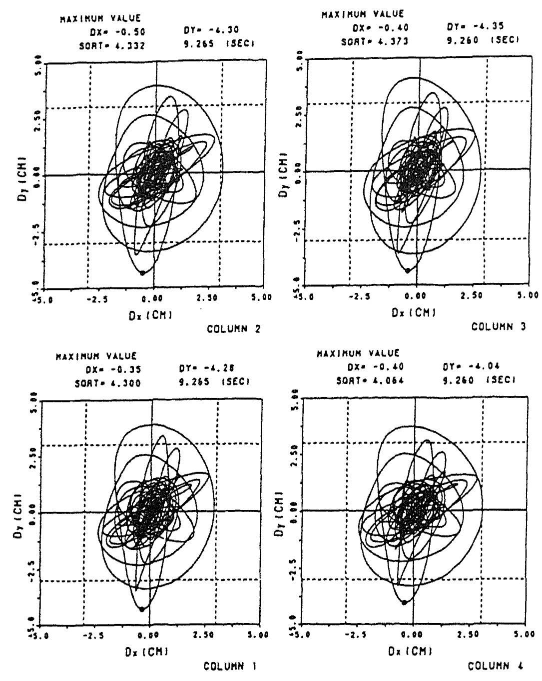

Figure 3. Displacement trajectories of C1 structure

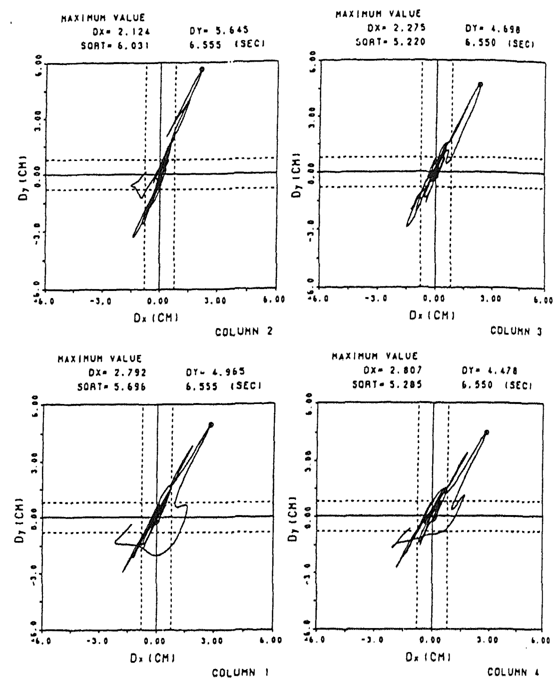

Figure 5. Displacement trajectories of C3 structure

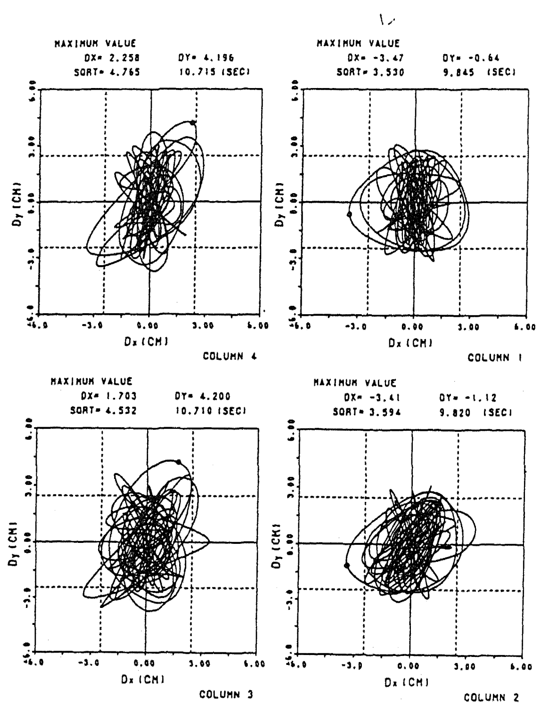

Figure 4. Displacement trajectories of B4 structure

motions may occur, when the root mean square of the maximum acceleration responses along the individual components has just arrived at the value where the identical structure has become collapsed under single component of the excitation.

4 CONCLUSIONS

From the experimental evidence, the following conclusions can be drawn:

1. Aseismic safety of the weak beam type structure can be estimated by assessing reserve strength against low cycle fatigue failure of the beam, without any consideration on the direction of horizontal earthquake, if the beam would efficiently stiffen against buckling.
2. The weak column type structure may sustain severe damage, which lead to the overall collapse due to P-Δ effect, under the two horizontal components of earthquake. However, the interaction effect may be insignificant, when the structure could behave slightly beyond the elastic limit.

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