DYNAMIC RESPONSE OF STEEL SPACE FRAMES UNDER EARTHQUAKE EXCITATION IN HORIZONTAL ARBITRARY DIRECTION

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Summary A dynamic analysis of the elasto-plastic responses of one-story single-bay space frames is performed. Results show that the displacement responses under the two-directional excitation become larger than those under the one-directional excitation.

INTRODUCTION

When a three-dimensional structure is subjected to earthquake excitation in an arbitrary direction, effects of biaxial bending generally appear on the inelastic behavior of structural members, and the dynamic behavior may be quite different from that of a plane frame under the in-plane excitation. From this point of view, this paper presents the results of analysis of the three-dimensional static behavior of an elasto-plastic cantilever beam-column, and of the dynamic behavior of one-story single-bay space frames.

STATIC ANALYSIS OF H-SHAPED BEAM-COLUMN UNDER HORIZONTAL FORCES

Analytical Model and Assumptions Analytical model is a cantilever column subjected to the constant vertical and varying horizontal forces at the free end, as shown in Fig. 1. Horizontal forces are computed for a given value of the column top displacement, taking the P-Δ effect into account, based on the following assumptions: 1) Deformable portion is concentrated near the column base with a certain length, in which curvature is constant. 2) Plane remains plane after the deformation takes place. 3) Stress-strain relation of the material is bi-linear as shown in Fig. 2. 4) Constant stress is distributed within the area of a subdivided segment of the cross section. 5) Column deforms without twist.

Results and Discussion Figure 3 shows an example of the restoring force characteristics of an H-shaped beam-column (H-300x300x10x15) under the horizontal forces applied in such a way that the displacements follow the prescribed circular path. Definitions of P, Q_x, Q_y, u and v are given in Fig. 1, and the values of these quantities at the initial yield when each of the forces acts alone are denoted by P_y, Q_{xy}, Q_{yy}, u_y and v_y, respectively. λ_x is the slenderness ratio about the strong axis of the H-shape. Figure 3 shows quite different characteristics compared with the case of the in-plane loading; two peaks in the virgin loading and the slip type hysteresis loops in Fig. 3(a), and the vertical slope of the loop in the vicinity of the displacement reversal in Fig. 3(b). In Q_y/Q_{yy}-v/v_y relations, the area enclosed in one loop increases due to the strain-hardening, but it seems on the contrary in Q_x/Q_{xy}-u/u_y relations. These phenomena are all due to the effect of biaxial loading. This analysis is performed to investigate the effects of biaxial bending, and used in determining the restoring force in the following dynamic analysis of space frames.

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DYNAMIC ANALYSIS OF ONE-STORY SINGLE-BAY SPACE FRAMES

Analytical Models Analytical models are two types of unbraced and braced space frames as shown in Fig. 4. Type II has an identical stiffness in x- and y-directions, and braces of Type III are designed so that the ultimate strengths under the static loading become nearly identical in the two directions, neglecting compression braces. Mass of the frame is concentrated at the center of the rigid horizontal roof. Ground acceleration is sinusoidal with the initial value equal to zero, of which frequency is 0.8 times the natural frequency of the frame in x-direction in case of Types I and II, and 0.6 times that in y-direction in case of Type III. Types I and II are subjected to three waves of ground motion, and the duration for Type III is ten seconds. Damping factors in x- and y-directions are both taken equal to 0.01, and natural periods T_x, T_y, T_θ are shown in Tables 1 and 2. The equation of motion of the system is solved by Newmark's β-method, taking β equal to 0.25. The restoring force of the column is determined in a manner described above in the analysis of the column, considering the change in axial force, while that of the brace is determined from the formulae in Ref.[1].

Results and Discussion Tables 1 and 2 show the maximum responses, where asterisks indicate the response in either x- or y-direction reaching ±30, at which the computation is terminated by considering that the frame has failed. In Type I, the maximum displacements of frames under the two-directional excitation are generally greater than those under the one-directional excitation, some of the former failing (marked by asterisks), while the latter being still stable. This is due to the interaction of biaxial bending. The maximum responses of Type I under the two-directional excitation increase with the increase in λ_x, AR, and P/P_y. This tendency may be observed in Type II, but however it seems that they are more stable than the frames Type I, since the frames Type II have identical stiffness and strength in x- and y-directions. Figures 5(a) to (d) show an example of an unbraced frame that is stable under one-directional excitation (Figs. 5(c),(d)), but fails under the two-directional excitation with the displacement u/u_y becoming enormously large (Figs. 5(a),(b)). Note that Q_y/Q_{yy}-v/v_y loops seem still stable. The failure of a plane frame is quite clearly defined as a point of zero restoring force with the negative stiffness. This definition may not be directly applicable to the space frame having the behavior such as shown in Figs. 5(a) and (b). Figs. 6(a) and (b) show the behavior of a braced frame failing under the two-directional excitation, the point of zero restoring force with negative stiffness appearing on both Q_x/Q_{xy}-u/u_y and Q_y/Q_{yy}-v/v_y loops in this case. The restoring force characteristics of the brace itself under the repeated axial loading clearly appear in Figs. 6(a) and (c).

CONCLUSION

Dynamic analysis of the elasto-plastic responses of one-story single-bay space frames is performed, and it is shown that displacement responses under the two-directional excitation are generally larger than those under the one-directional excitation, which lead to an earlier collapse. A careful check is thus needed in designing earthquake resisting frames.

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REFERENCE

[1] M.Wakabayashi, T.Nakamura, et al., 1977, "Hysteretic Behavior of Steel Braces Subjected to Horizontal Load due to Earthquake", Proc.6th WCEE, New Delhi, Vol 3, PP. 3188-3194

[2] C.Matsui, S.Morino, Y.Uchida, 1979, "Dynamic Response of Steel Space Frames Under Earthquake Excitation in Horizontal Arbitrary Direction", Proc. Annual Meeting of Architectural Inst. of Japan, PP. 1015-1016

Table 1 Maximum Responses of Unbraced Space Frames (Types I, II)

Type	λx	P/Py	ARx,ARy	(u/u_y)max	(v/v_y)max	θmax($\times10^{-3}$)	($\bar{u}$/u_y)max	($\bar{v}$/v_y)max	Tx	Ty	Tθ	Cbx	Cby
I	20	0.1	1.0	-4.71	3.57	-0.84	3.84	-3.23	0.34	0.20	0.14	0.30	0.87
			1.5	-13.14	-8.23	-3.44	5.18	-7.90					
		0.3	1.0	-7.45	-4.16	-1.30	4.03	-4.09	0.60	0.35	0.25	0.10	0.29
			1.5	-30.00*	-11.35*	-5.72 *	5.21	-10.78					
		0.5	1.0	-18.90	-7.50	-9.53	3.36	-8.08	0.77	0.45	0.32	0.06	0.17
	30	0.3	1.0	-30.00*	-5.96 *	-4.06*	5.75	-4.64	1.09	0.64	0.45	0.07	0.19
	40	0.072	1.0	-8.24	4.22	-6.15	3.92	-3.19	0.83	0.48	0.34	0.20	0.60
		0.1	1.0	-15.78	5.14	-5.48	4.19	-3.35	0.97	0.57	0.40	0.15	0.44
			1.5	-30.00 *	-8.79 *	-10.41*	6.62	-8.46					
		0.3	1.0	-30.00 *	-8.02 *	-4.77 *	-30.00 *	-9.46	1.69	0.98	0.69	0.05	0.15
			1.5	-30.00 *	-14.54 *	-12.26*	30.00 *	30.00 *					
	80	0.036	1.0	-30.00 *	5.21 *	22.77 *	5.12	3.33	1.65	0.96	0.68	0.20	0.60
II	30	0.3	1.0	-3.28	-3.24	5.24	—	—	0.78	0.78	0.45	0.13	0.13
	40	0.3	1.0	-9.30	-8.58	18.26	—	—	1.20	1.20	0.69	0.10	0.10

ARx, ARy = ratios of the intensity of excitation to the initial yield strength of the frame under static horizontal force in x- or y-direction respectively; u, v = displacements of the roof center in x- and y-directions respectively; $\bar{u}$, $\bar{v}$ = values of u and v under the one-directional ground motion respectively; θ = angle of rotation of the roof; Tx, Ty, Tθ = natural periods in the sway modes in x- and y-directions, and in the rotational mode respectively; C_{bx}, Cby = base shear coefficients.

Table 2 Maximum Responses of Braced Space Frames (Type III)

Type	λx	P/Py	ARx,ARy	(u/u_y)max	(v/v_y)max	θmax($\times10^{-2}$)	($\bar{u}$/u_y)max	($\bar{v}$/v_y)max	Tx	Ty	Tθ	Cbx	Cby	λ_b
III	20	0.3	1.0	5.71	3.58	-2.82	4.62	3.63	0.38	0.35	0.21	0.19	0.29	148
	40	0.132	1.0	-8.41	5.30	-3.32	-8.71	5.59	0.50	0.65	0.32	0.22	0.33	150
		0.3	1.0	-46.92	-22.72	-26.41	6.77	-15.60	0.86	0.98	0.53	0.07	0.15	150
	80	0.066	1.0	-11.23	6.88	-0.38	-2.18	6.53	0.72	1.30	0.52	0.21	0.33	150

λ_b = slenderness ratio of the brace.

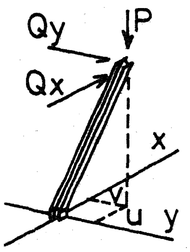


Fig.1 Cantilever Beam-Column

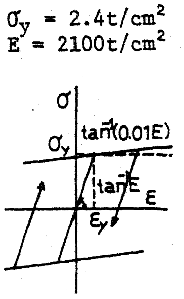
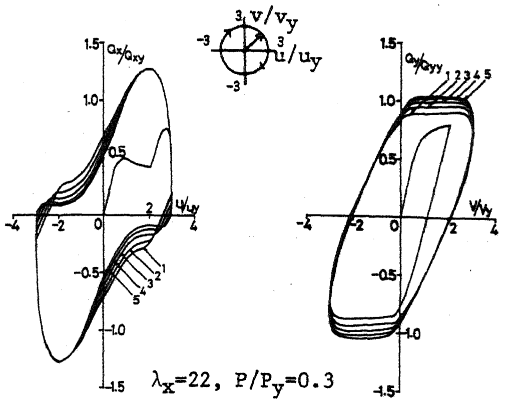


Fig.2 Stress-Strain Relation



(a) (b)

Fig.3 Restoring Force Characteristics of Cantilever Beam-Column

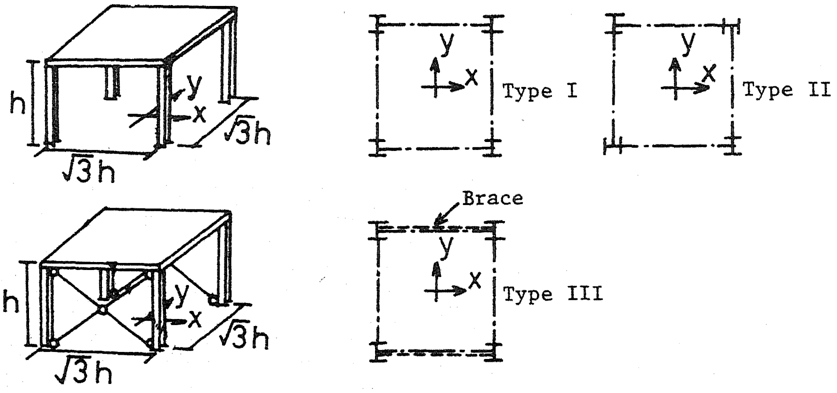
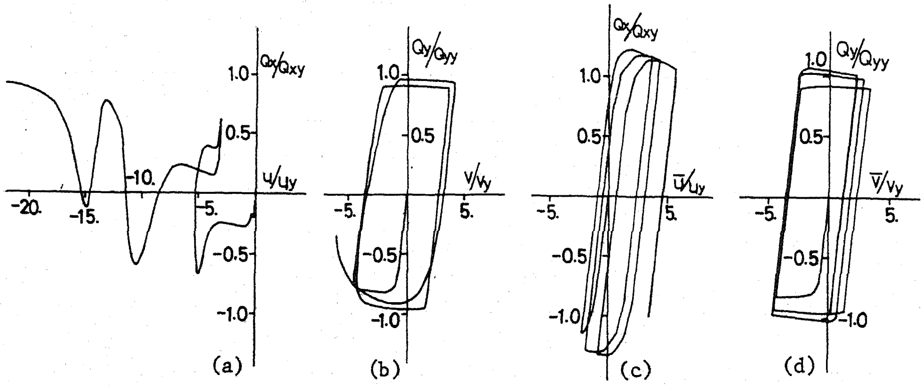
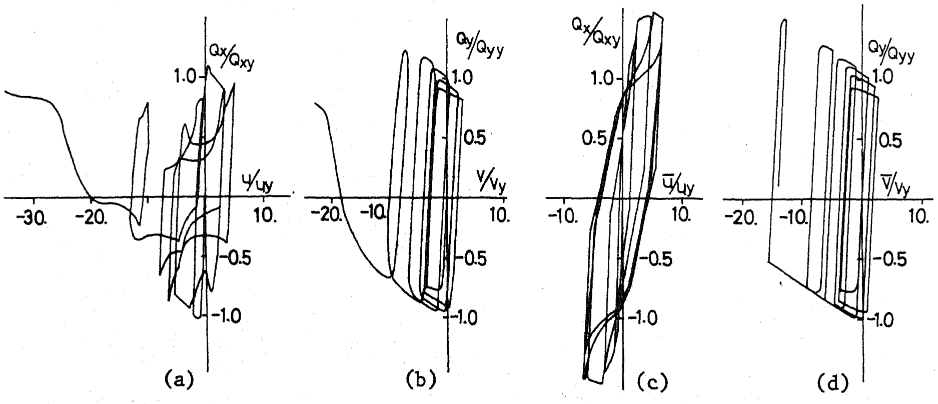


Fig.4 Braced and Unbraced Space Frames



Type I: $\lambda_x=30$, $P/P_y=0.3$, $AR_x=AR_y=1.0$

Fig.5 Restoring Force-Displacement Relation of Unbraced Space Frame



Type III: $\lambda_x=40$, $P/P_y=0.3$, $AR_x=AR_y=1.0$

Fig.6 Restoring Force-Displacement Relation of Braced Space Frame