SEISMIC RESPONSE OF SIX-STORY ECCENTRICALLY BRACED STEEL FRAMES WITH COLUMNS PARTIALLY ALLOWED TO UPLIFT

M. Midorikawa[1], S. Toyomaki[2], H. Hori[3], T. Asari[4], T. Azuhata[5] and T. Ishihara[6]

[1] *Professor, Graduate School of Engineering, Hokkaido University, Sapporo, Japan*
[2] *Takenaka Corporation (Former Graduate Student of Hokkaido University), Nagoya, Japan*
[3] *Graduate Student, Graduate School of Engineering, Hokkaido University, Sapporo, Japan*
[4] *Assistant Professor, Graduate School of Engineering, Hokkaido University, Sapporo, Japan*
[5] *Head, National Institute for Land and Infrastructure Management, Tsukuba, Japan*
[6] *Senior Researcher, National Institute for Land and Infrastructure Management, Tsukuba, Japan*
Email: midorim@eng.hokudai.ac.jp, toyomaki.shingo@takenaka.co.jp, taikenhirotake@yahoo.co.jp, asari@eng.hokudai.ac.jp, azuhata-t92ta@nilim.go.jp, ishihara-t92hd@nilim.go.jp

ABSTRACT :

Previous studies have suggested that rocking vibration accompanied with uplift motion might reduce the seismic damage of buildings subjected to strong earthquake motions. In this paper, the seismic response of eccentrically braced steel frames with partial columns allowed to uplift is evaluated and compared with that of fixed-base frame by numerical analyses, whose prototype test frame of a full-scale six-story eccentrically braced steel building structure was tested by pseudo-dynamic testing technique in Tsukuba in 1982 to 1984. The results show that the base shears in the column-uplift frames are significantly reduced as compared to the fixed-base frames, while the maximum roof drifts of the uplift models are larger than those of the fixed-base models regardless of the input motion intensity because the rotational rigidity of the uplift-column bases in the uplift model decreases.

KEYWORDS: inelastic seismic response analysis, braced steel frame, rocking vibration, column uplift, yielding base plate

1. INTRODUCTION

It has been pointed out by past studies (Housner 1963, Rutenberg et al. 1982, Hayashi et al. 1999) that the effects of rocking vibration accompanied with uplift motion might reduce the seismic damage to buildings subjected to strong earthquake ground motions. The influence of uplift motion on the seismic behavior of building structures has been reasonably explained through simple analyses (Meek 1975, 1978). On the other hand, it has been shown that there is a scale effect in the rocking oscillation which makes the larger of two geometrically similar blocks/structures more stable than the smaller block/structure (Kobori and Minamii 1956, Housner 1963, Oliveto et al. 2003), and that the overturning collapse may not be caused in rigid buildings against strong earthquake motions (Tagawa et al. 2003).

On the basis on these studies, structural systems have been studied and developed that allow rocking vibration and uplift motion under proper control during strong earthquake motions (Clough and Huckelbridge 1977, Huckelbridge 1977, Kasai et al. 2001, Iwashita et al. 2002, Midorikawa et al. 2006a). One of the features of an uplift system is that the maximum strain energy associated with the horizontal deformation of a superstructure is reduced because a portion of the total earthquake input energy exerted to a structural system is dissipated by the potential and kinetic energy associated with vertical motion of a system as shown in the work by Iwashita et al. (2003) and Azuhata et al. (2004). When an uplift structural system is applied to a building, several advantages are expected in the seismic design such that these systems can lead to reduced seismic response, which will lead to a more rational and economical seismic design of not only structural elements but also foundations.

An uplift structural system under research and development by the authors (Azuhata et al. 2004, Midorikawa et al. 2006a, 2006b) makes use of the uplift yielding mechanism of flexible base plates. When flexible base plates yield due to column tension during a strong earthquake motion, the columns uplift and allow the building structure to rock. In the previous work by the authors, the seismic behavior of steel frames with all uplift columns have been studied but not those with partially uplift columns.

In this paper, presented are the comparison of the seismic behavior of the analytical results of six-story braced-steel structures with fixed base and partially uplift columns, and also the evaluation of analytical models as compared with the full-scale seismic tests on an eccentrically braced steel structure (Roeder 1989, Foutch 1989), which were carried out in Tsukuba in early 1980's. The objectives of the study are to improve the understanding of the dynamic response of a structure with partially uplift columns subjected to earthquake motions, and to validate analytical models to simulate the response of partial rocking through column uplift.

2. FULL-SCALE SEISMIC TEST STRUCTURE

The test structure was originally designed as a six-story concentrically braced-steel building with fixed base conforming to the seismic provisions of Japan and U. S. for Phase I test program (Roeder 1989). It consisted of a two-by-two bay structure. Frames A, B and C in the test direction were moment resisting frames, while the exterior frames (Frames 1 and 3) in the transverse direction had cross-bracing with simple girder connections. An eccentric bracing system was installed in the center frame (Frame B) following the removal of the concentric braces from Frame B upon completion of Phase I test. The design and construction of the test structure, test program, test procedure, and instrumentation are described in detail in other publication (Foutch 1989).

In the seismic tests, the simulated seismic forces by the pseudodynamic test system were applied. The response of the structure to the 1952 Taft S21°W record scaled to 65cm/s^2 for the elastic test and to 500cm/s^2 for the inelastic test was simulated. Since relatively small damage was observed in the test structure during the inelastic test, a series of three tests using sinusoidal input motions were carried out. In the third sinusoidal test, the gusset plates connecting the second-floor (2FL) shear link to the braces buckled causing the large inelastic torsional deformation of the girder and the degradation of the strength and stiffness of the first story.

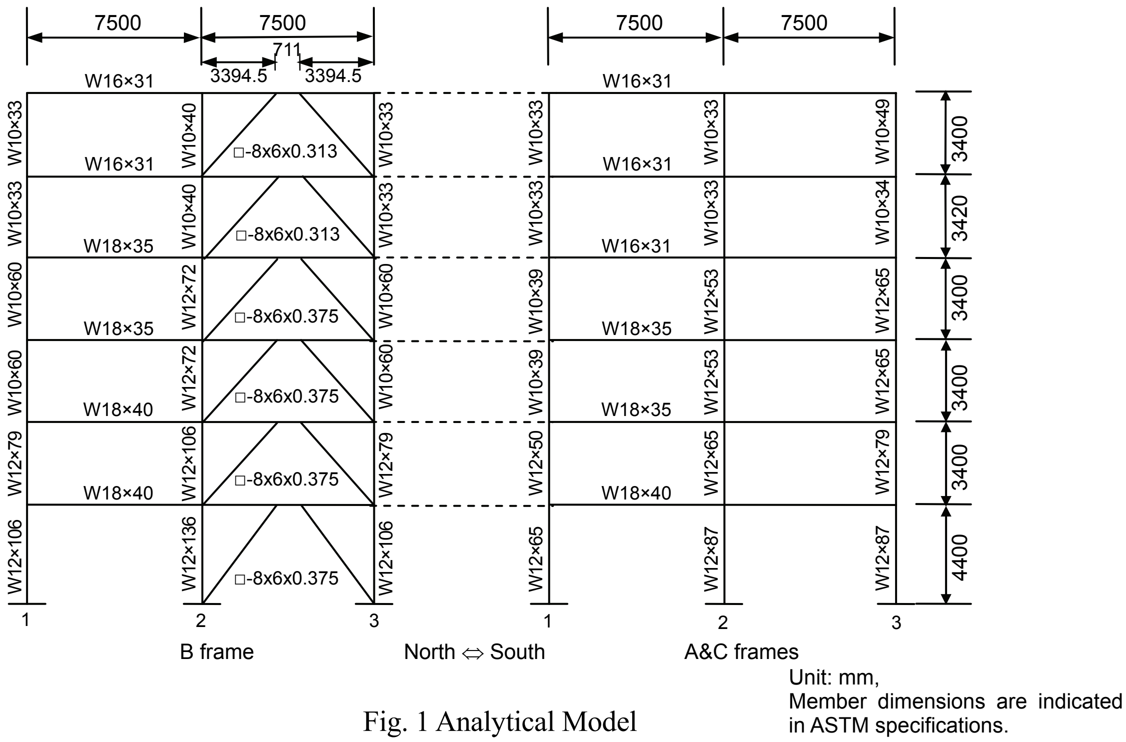


Fig. 1 Analytical Model

3. ANALYTICAL MODELLING AND NUMERICAL ANALYSES

3.1 Fixed-base Model

The modified version of the two-dimensional nonlinear dynamic analysis program DRAIN-2DX (Kanaan and Powell 1973) was used in the analyses. The mathematical idealization of the test structure is illustrated in Fig. 1. The three frames (Frames A, B and C) was reduced to two frames because of the symmetry of the structure in the test direction. All column bases of this model were fixed and referred to as Fixed-base model. Two element types, beam-column and beam elements, were used to model the columns, composite girders, braces, and shear links located at midspan of the girders in the braced bay. The main assumptions used to model the test structure are followed by the previous work (Midorikawa et al. 1989) except for the assumption that the restraint on axial deformation of columns by diagonal cross-bracing and girders in the transverse direction were neglected. It is assumed that the viscous damping resulted from a combination of the mass- and initial stiffness-dependent effects (Rayleigh-type). The critical damping ratios of 2% were introduced to the first two modes referring to the previous work (Goel 1989). The numerical time integration in the analyses was the Newmark method with the integration constant of 0.25, based on a constant acceleration with time steps of 0.005 seconds.

3.2 Uplift Base-plate Yielding Model

The main assumptions used to model the structure with partially uplift columns and yielding base-plates, referred to as BPY model, were the same as Fixed-base model except for the following: The yielding base-plates to allow the column uplift as shown in Figs. 2(a) and (b) were assumed to be installed at the bottom of two columns of the braced bay at the first story. The base-plate configuration was determined so that its uplift yield strength was approximately 10% of the column axial load. The yield strength and elastic stiffness of the base-plate, assuming to be mild steel of yield strength of 294MP, were estimated following the previous works

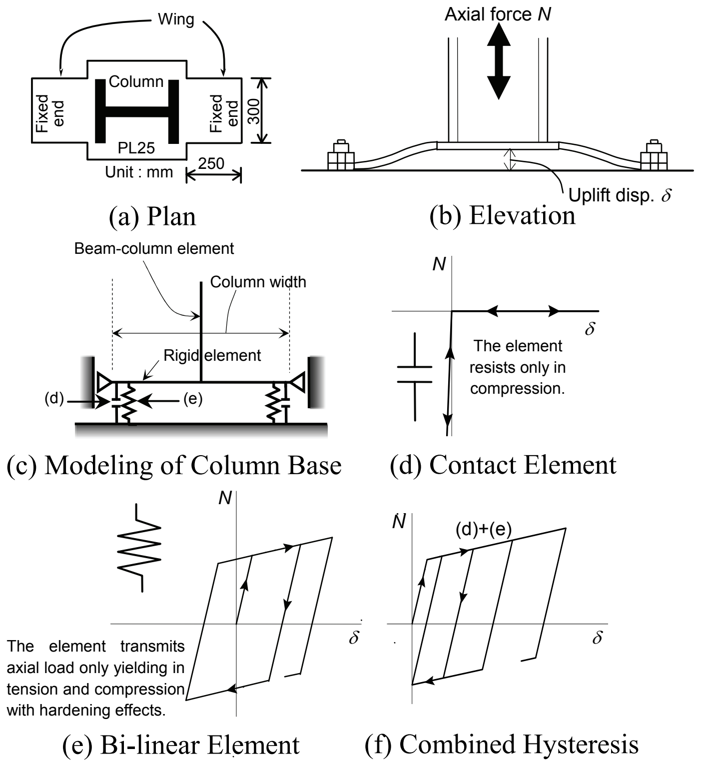


Fig. 2 Uplift Yielding Base-plate

Table 1 Earthquake Ground Motions

Seismic motion	Year	PGA (cm/s²)	PGV (cm/s)
Taft S21°W	1952	173	15.4
El Centro NS	1940	342	33.1
JMA Kobe NS	1995	818	84.7

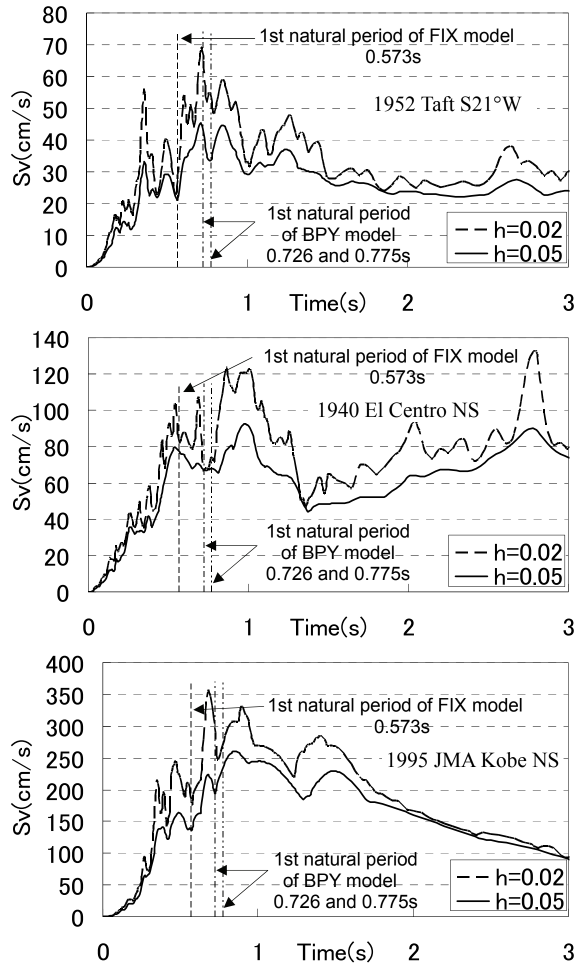


Fig. 3 Velocity Response Spectra

(Sudo et al. 2006, Midorikawa et al. 2002, Ishihara et al. 2003). Fig. 2(c) shows the mathematical model of yielding base-plate representing its deformation mechanism as shown in Fig. 2(b). This model has two sets of the parallel combination of contact and bi-linear springs, as shown in Figs. 2(d) and (e). Fig. 2(f) shows the combined hysteresis model of yielding base-plate assuming the kinematic hardening scheme whose strain hardening stiffness was 15% of the initial elastic stiffness, referring to the previous study (Sudo et al. 2006).

3.3 Earthquake Ground Motions

Three sets of earthquake ground acceleration records listed in Table 1 were selected for the response analyses. The elastic velocity response spectra of three input earthquake motions are shown in Fig. 3. The maximum ground velocity of each record with the duration time of 20 seconds was scaled to 30 through 150 cm/s.

4. RESULTS AND DISCUSSION

4.1 Natural Periods and Pushover Analyses

The natural periods from the analytical and test results of Fixed-base model are listed in Table 2. The fundamental natural period of Fixed-base model is about 5% longer than that of the test structure, and that of BPY model is 0.726 and 0.775 s on the assumption that one of two uplift-column bases is a pin support.

Table 2 Natural Periods of Fixed-base Model

Mode	1st	2nd	3rd
Model (s)	0.573	0.207	0.115
Test (s)	0.545	0.193	0.106

Fig. 4 shows the story shear versus interstory drift relations at each floor level from the pushover analysis for Fixed-base model under the A_i distribution of story shears along the height. The story shear versus roof drift relations from the pushover analyses for Fixed-base and BPY models are shown in Fig. 5. The mark '*' in the figure indicates that the 2FL shear link (SL) reaches the ultimate shear deformation of 0.09 rad. corresponding to the maximum shear strength of the shear link observed in the seismic tests (Foutch 1989).

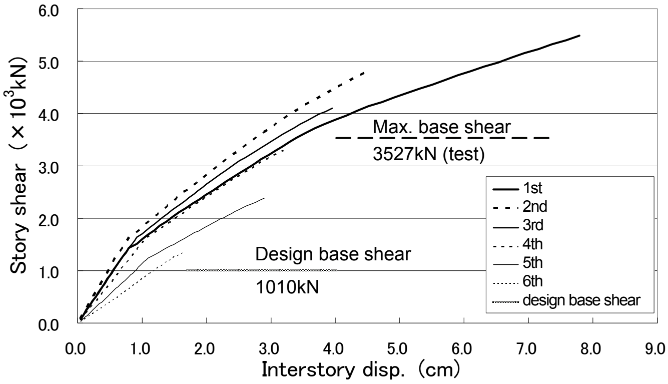


Fig. 4 Story Shear vs. Interstory Drift Relations

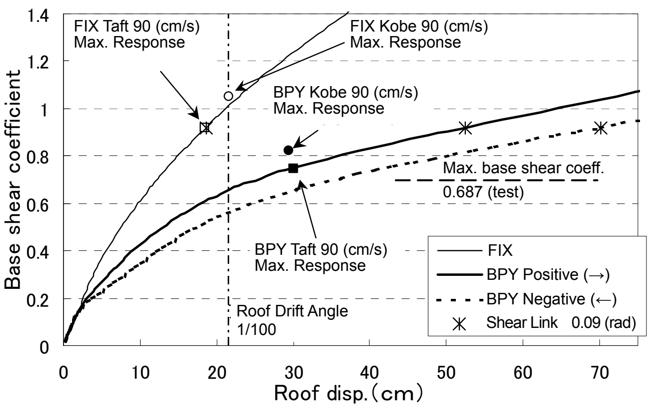


Fig. 5 Base Shear Coefficient vs. Roof Drift Relations

4.2 Seismic Responses of Fixed-base Model

4.2.1 Comparison of fixed-base model and test structure

The time histories of the roof drifts of BPY model and the test structure subjected to Taft record of 65 and 500cm/s^2 respectively are shown in Fig. 6. The maximum roof drifts between the analysis and test do not match well in the elastic response because of the slight difference of fundamental natural periods, while they harmonize well with each other in the inelastic response. Fig. 7 shows the shear force versus shear deformation relations of the 2FL SL to Taft record of 500cm/s^2. The maximum deformation from the analysis is slightly smaller than that from the test. The shear link attains the ultimate shear strength at the shear deformation of 0.09 rad. during the sinusoidal tests. It is reported in the previous test (Popov et al. 1987) that the ultimate and design-limit shear deformation of a fully stiffened shear link are 0.10 and 0.06 rad., respectively.

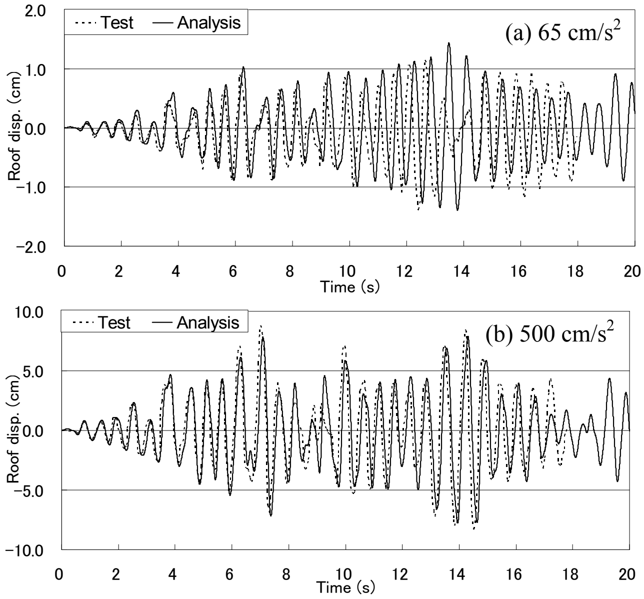


Fig. 6 Horizontal Roof Drift Time Histories: Taft S21°W

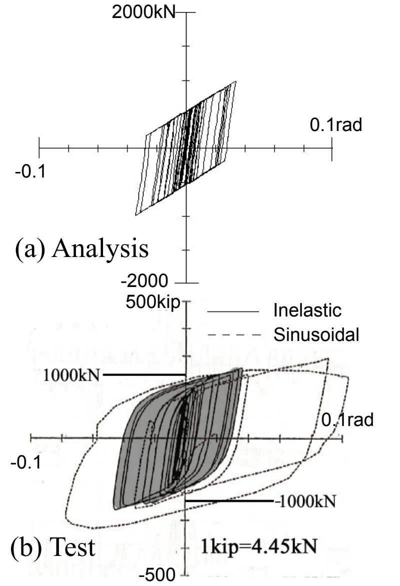


Fig. 7 Shear Force vs. Shear Deformation Relations of 2nd-floor Shear Link

4.2.2 Maximum responses versus input ground velocity

The maximum shear deformation versus input ground velocity relations of Fixed-base model are shown in Fig. 8, and the maximum roof drift versus input ground velocity relations in Fig. 9. In the figures, the test-ultimate and design-limit shear deformations of the 2FL SL and the corresponding maximum input velocity are indicated by the broken and dotted lines, respectively. The maximum roof drift to Taft record of 500cm/s^2 is 1/272 (7.9 cm), while it is 1/108 (19.8 cm) when the shear deformation of 2FL SL reaches the ultimate value of 0.09 rad.

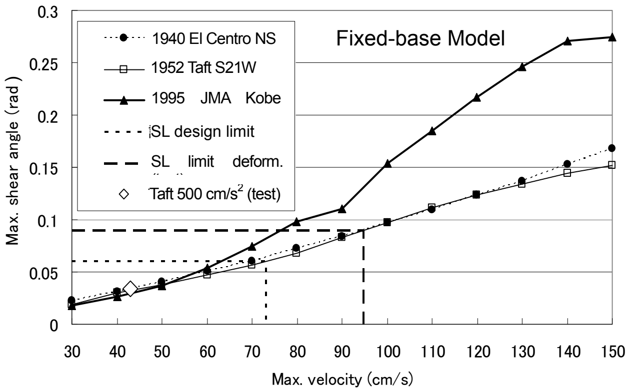


Fig. 8 Maximum Shear Deformation vs. Input Velocity Relations of 2nd-floor Shear Link

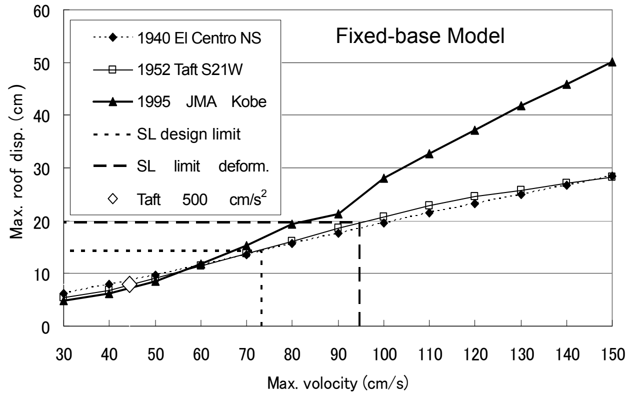


Fig. 9 Maximum Roof Drift vs. Input Velocity Relations

4.3 Comparison of Seismic Responses of Fixed-base and BPY Models

4.3.1 Time histories of roof drifts and uplift displacements

The roof drift time histories of Fixed-base and BPY models to Taft record of 90cm/s are shown in Fig. 10, and the uplift displacement time histories at the column bases of the braced bay of BPY model in Fig. 11. The roof drift of BPY model is considerably larger than that of Fixed-base model because of the reduction of the rotational stiffness at the column bases in BPY model. The maximum uplift displacement of BPY model is 8.56 cm that corresponds to the rocking angle of 1/88 in the braced bay.

4.3.2 Maximum responses versus input ground velocity

The maximum uplift displacement at the column bases of the braced bay of BPY model versus input ground velocity relations are shown in Fig. 12. The maximum uplift displacements to JMA Kobe record of 90cm/s are 10.4 and 8.35 cm at the inner and outer column bases, respectively, that correspond to the rocking angle of 1/72 and 1/90 in the braced bay, respectively. The maximum roof drift versus input ground velocity relations of Fixed-base and BPY models are shown in Fig. 13. The roof drift of BPY model is considerably larger than that of Fixed-base model. The maximum base shear coefficient versus input ground velocity relations of Fixed-base and BPY models are shown in Fig. 14. The base shear coefficient of BPY model is limited to a constant value over the input velocity of about 90cm/s, while that of Fixed-base model increases monotonously.

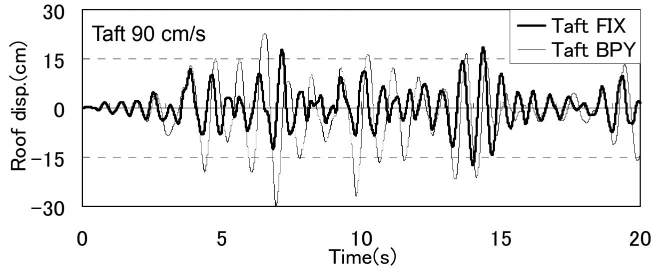


Fig. 10 Horizontal Roof Drift Time Histories

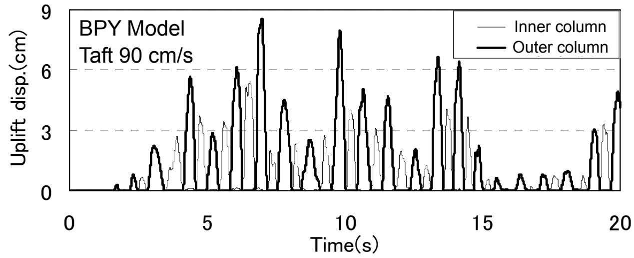


Fig. 11 Uplift Displacement Time Histories at Column Bases

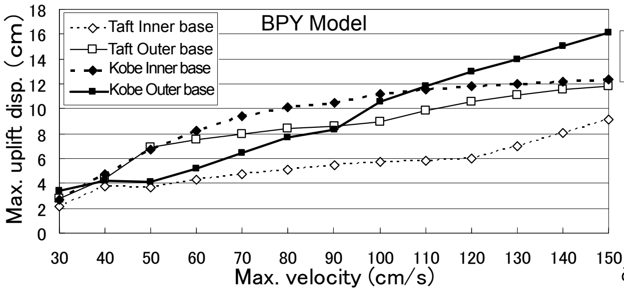


Fig. 12 Maximum Uplift Displacement vs. Input Velocity Relations at Column Bases

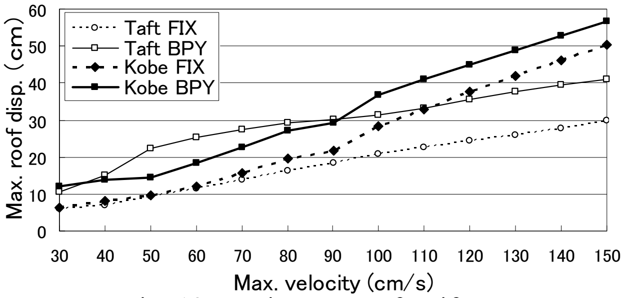


Fig. 13 Maximum Roof Drift vs. Input Velocity Relations

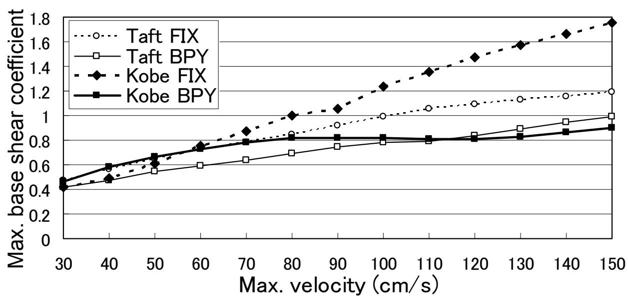
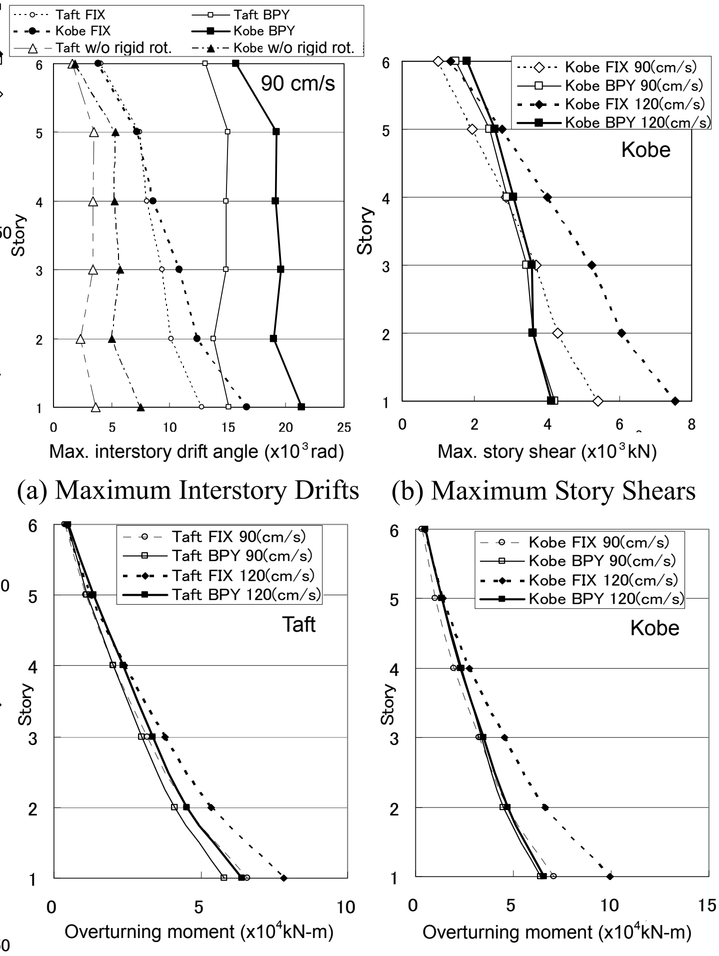


Fig. 14 Maximum Base Shear Coefficient vs. Input Velocity Relations



(a) Maximum Interstory Drifts (b) Maximum Story Shears

(c) Maximum Overturning Moments

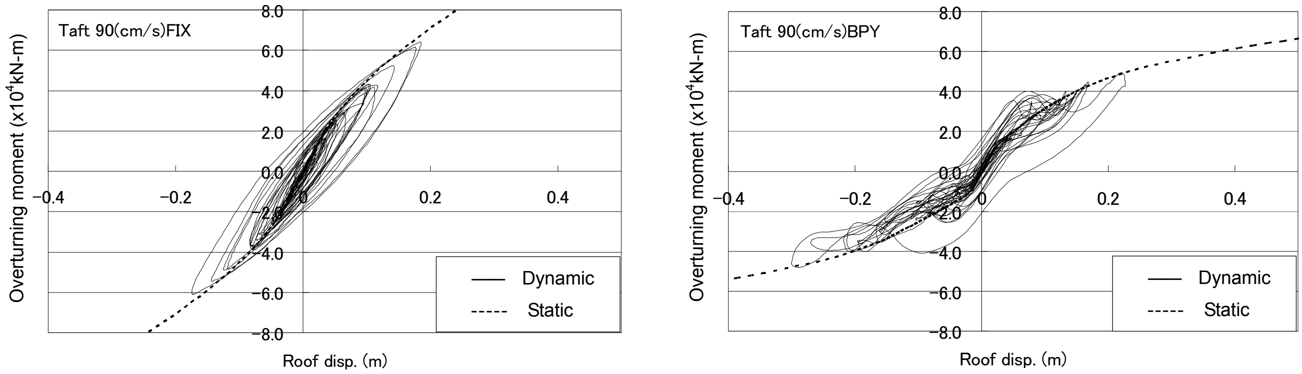
Fig. 15 Envelopes of Maximum Responses

4.3.3 Envelopes of maximum responses

The envelopes of the maximum interstory drifts, story shears and overturning moments of Fixed-base and BPY models are shown in Fig. 15. The maximum interstory drifts of BPY model to Taft and JMA Kobe records of 90cm/s are much larger than those of Fixed-base model as shown in Fig. 15(a). As a result, the inelastic deformation of structural members in the moment-resisting frames of BPY model becomes larger than that of Fixed-base model. The maximum story shears at the lower stories of BPY model are definitely smaller than those of Fixed-base model, while they become nearly equal at the upper stories, as shown in Fig. 15(b). The overturning moments (OTM) of BPY model are certainly smaller than those of Fixed-base model as shown in Figs. 15(c). There is no significant difference between the input velocity of 90 and 120cm/s in OTM of BPY model, while there is a definite difference in OTM of Fixed-base model. This is compatible with that the base shear coefficient of BPY model is limited to a constant value over the input velocity of about 90cm/s.

4.3.4 Hysteretic behavior of Fixed-base and BPY models

The base OTM versus roof drift relations to Taft record of 90cm/s are shown in Fig. 16. The base OTM versus roof drift relations from the pushover analyses are plotted in dotted lines in the figure. The envelopes of these relations from the dynamic responses correspond fairly well with those from the static analyses. The restoring force versus deflection relation of Fixed-base model remains quite stable, whereas that of BPY model shows the attribute of the fluctuation in the restoring force. This is probably the effect of higher mode vibration induced in an uplift structure as pointed out by the previous studies (Meek 1975, 1978, Ishihara et al. 2006).



(a) Fix-base Model: Taft S21°W; 90 cm/s (b) BPY Model: Taft S21°W; 90 cm/s

Fig. 16 Base Overturning Moment vs. Roof Drift Relations

5. SUMMARY AND CONCLUSIONS

The seismic response of braced-steel structures with partially uplift columns are evaluated and compared with that of fixed-base structures by numerical analyses. The studies are carried out using six-storey, 2×2 bay eccentrically braced full-scale steel frames, whose prototype structure was tested in early 1980's. The results of this study are summarized below:

(1) The analytical results of the fixed-base model subjected to Taft record of 500cm/s^2 show good agreement with the inelastic test results of the full-scale test structure. In addition, the seismic behavior of the full-scale test structure over the seismic intensity level applied in the tests is validated from the analyses, which was not accomplished in the tests.

(2) The base shears in the uplift-column model are significantly reduced as compared to the fixed-base model, whereas the maximum roof drifts of the uplift-column model are larger than those of the fixed-base model regardless of the input motion intensity because of the reduction of the rotational rigidity of the uplift-column bases. As a result, the inelastic deformation of structural members of the uplift-column model becomes larger than that of the fixed-base model.

The results discussed above regarding the behavior of partial-column uplift systems are not directly relevant to the practical design of structures. Further study is needed to establish the optimal range of design parameters of partial-column uplift systems to achieve the desired system performance under various earthquake intensities. The research project on a large-scale test of a rocking braced-frame fuse system at the E-Defense facility in Japan is scheduled for 2009 in collaboration with researchers and engineers in Japan and U. S. This project is expected to further develop, improve, and validate the seismic performance of rocking systems.

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