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Dynamic Instability in High-Rise SMRFs Subjected to Long-Period Ground Motions

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***Abstract*—**We study dynamic instability in high-rise steel moment resisting frames (SMRFs) subjected to synthetic long-period ground motions caused by hypothetical huge subduction earthquakes. Since long duration as well as long dominant periods is a characteristic of long-period ground motions, interstory drifts may enter the negative postyield stiffness range many times when high-rise buildings are subjected to long-period ground motions. Through the case studies of 9 high-rise SMRFs designed in accordance with the Japanese design practice in 1980s, we demonstrate that drifting, or accumulation of interstory drifts in one direction, occurs at the lower stories of the SMRFs, if their natural periods are close to the dominant periods of the long-period ground motions. The drifting led to residual interstory drift ratio over 0.01, or to collapse if the design base shear was small.

***Keywords*—**long-period ground motion, P-Delta effect, high-rise steel moment resisting frame (SMRF), subduction earthquake

I. Introduction

THERE is a growing concern that huge subduction earthquakes may take place in the Pacific coast near Japan in the next few decades [1-3]. When a huge subduction earthquake strikes Japan, metropolitan areas like Tokyo, Nagoya, and Osaka may experience ground motions having very strong components in the long-period range. In the metropolitan areas, high-rise buildings are among the structures most vulnerable to such ground motions. Since long duration is another characteristic of such ground motions, high-rise buildings may experience many cycles of large interstory drift [1]. This concern led to the research focusing on (1) low- to moderate-cycle fatigue resistance of structural members and connections [1,4,5] and (2) in-room safety against the shaking of furniture and non-structural components at the upper floors [6,7]. In fact, non-structural components of many high-rise building located in Tokyo and Osaka were severely damaged in the great Tohoku earthquake on March 11, 2011.

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Although structural damage due to dynamic instability is of another serious concern, the research addressing this issue has been scarce. It should be noted that, in this paper, only the ground motions induced by subduction earthquakes are called as long-period ground motions, although the ground motions caused by near-fault earthquakes may also have long dominant periods [8-12].

On dynamic instability of building structures, a number of research papers have been published so far [13]. For a single-degree-of-freedom system, the mechanism of dynamic instability is well known [14]. Drifting, or accumulation of deformation in one direction, occurs when the interstory drift enters the negative postyield stiffness range. The system collapses when the deformation exceeds the limit at which the system loses its lateral resistance. The P-Delta effect plays key roles in dynamic instability. For a multi-degree-of-freedom system, dynamic instability occurs in a single story [15] or in a subset of stories [16] in accordance with the collapse mechanism. Uetani [17] and Uetani and Tagawa [18,19] pointed out that, when high-rise buildings are subjected to very strong ground motions, drifting takes place only at the lower stories of the building, where the influence of the P-Delta effect is high, even when the building is designed in accordance with the strong-column-weak-beam concept. Gupta and Krawinkler [20] and Yamazaki and Endo [21] reported similar results. Nevertheless, the response to long-period ground motions was not examined in these studies.

Only a few research works addressed dynamic instability of high-rise buildings subjected to long-period ground motions. Such research dates back at least to the investigation of the 22-story Pino Suarez building [22,23], which collapsed in the 1985 Michoacan, or Mexico City, earthquake. From the results of nonlinear time history analyses using plastic hinge models, it was pointed out that local buckling in columns as well as the P-Delta effect played key roles in the collapse of the building. Yang [24] studied the response of 20-story steel frames under long-period ground motions using detailed finite element models. Half of the models in the study were designed to satisfy the minimum requirements of the Japanese provisions [10,25]. Nevertheless, they do not reflect the design practice in Japan, where high-rise buildings are usually designed about 1.5 to 2 times stronger than the minimum requirements by increasing the design base shear. It is therefore still unclear how dynamic instability manifests in high-rise buildings designed in accordance with Japanese design practice when they are subjected to long-period ground motions.

The objective of this paper is to study how dynamic instability appears in high-rise steel buildings when they are subjected to long-period ground motions caused by hypothetical huge subduction earthquakes predicted to occur in the Pacific coast near Japan. In this paper, we focus on the response of steel moment resisting frames (SMRFs) designed in accordance with Japanese design practice during 1980s. This is because the number of existing high-rise buildings constructed in this period, most of which are SMRFs, is significantly higher than that in the other periods. Another reason is that the level of the design ground motions applied in this period was significantly smaller, especially in the long-period range over 3 sec, than those required in the current Japanese provisions [26-28].

Although the attention of this paper is focused on the high-rise SMRFs designed in accordance with Japanese design practice, the authors hope that our work motivates the compile of research efforts to reexamine the seismic resistance of high-rise buildings existing around the world to long-period ground motions caused by huge subduction earthquakes because more flexible and weaker SMRFs exist in many earthquake prone countries.

II. High-Rise Building Design in Japan

In this section, we provide a brief summary of the history of the structural design of high-rise buildings (taller than or equal to 60 m) constructed in Japan [1]. In Japan, construction of high-rise buildings started in 1963, when the restriction on the total height of buildings was removed from the Japanese building code. Existing high-rise buildings can be roughly classified into 3 groups in accordance with the period during which they were constructed: (1) the early period (1963-1980), (2) the middle period (1981-1995), and (3) the late period (1996-current). These periods are separated by the major revision of the Japanese building code in 1981 and the 1995 Hyogo-ken Nanbu, or Kobe, earthquake.

During the early period, most high-rise buildings were constructed as steel structures or steel reinforced concrete structures [30], which have lateral resisting elements like steel braces or reinforced concrete slit walls. In the time-history analysis, two levels of design ground motions were used. One level of ground motions, called Level 1 ground motions, are expected to occur several times during the service life of a building. No damage is allowed to the Level 1 ground motions. The other level of ground motions, called Level 2 ground motions, are expected to take place very rarely and considered to be the maximum a building possibly experiences in its service life. Collapse must be prevented to the Level 2 ground motions although partial damage is allowed. These design ground motions were obtained by scaling strong ground motion records, e.g., El Centro 1940, Taft 1952, and Hachinohe 1968 ground motions. At first, the strong ground motion records were often scaled so that their peak ground accelerations (PGAs) were 250 cm/sec^2 and 500 cm/sec^2 for Level 1 and Level 2 ground motions, respectively. With the advancement in time, the use of peak ground velocities (PGVs) became popular because PGVs are considered as a more robust indicator of the level of ground motions than PGAs. The PGVs for the Level 1 and Level 2 ground motions were 25 cm/sec and 50 cm/sec, respectively. It is worth noting that, in western areas of Japan, 20 % reduction was often applied to the design ground motions due to the low seismicity of the region. Since no strong motion record having large long-period components was available at the early stage of this period, the design base shear coefficient was recommended to be more than or equal to 0.05 regardless of the natural period of a building to prevent the design base shear from being too small in the long-period range [31].

Major revisions of the Japanese provision in 1981 resulted in significant changes in the design practice of high-rise buildings. During the middle period, SMRFs became most popular because absence of braces and walls allows flexible planning, and the revision also led to lower strength demands for SMRFs than those for other types of constructions. Although steel structures were still the most widely used in this period, the rate of reinforced concrete structures increased significantly. In order to increase the safety margin of high-rise buildings, the design base shear was usually increased about 1.5 to 2 times compared to the minimum demands in the revised Japanese provisions. From the late 1980s to the early 1990s, the Building Center of Japan (BCJ) developed Level 1 and Level 2 synthetic ground motions [32], which are called as BCJ L1 and BCJ L2 ground motions, respectively. The response spectra of the BCJ ground motions were defined at the engineering bedrock, and the amplification in soil should be considered appropriately at each site. In the long period range, the response velocity spectrum of the BCJ L2 ground motion has the constant value of 100 cm/sec. This value is much larger than those of the previously used Level 2 ground motions when the natural period is longer than 3 sec. The BCJ ground motions were increasingly used in the design of high-rise buildings and other long-period structures like base-isolated structures [33].

Experience of 1995 Hyogo-ken Nanbu earthquake made another significant impact on the design practice of high-rise buildings in Japan. After the 1995 Hyogo-ken Nanbu earthquake, the demands for higher seismic resistance led to the widespread use of energy dissipating devices like buckling restrained braces. In addition, from the late 1990s, construction of base-isolated high-rise buildings started. Sharp decrease was observed in the ratio of steel structures to other types of structures in high-rise buildings constructed in this period. Instead, the ratio of concrete filled tube structures and reinforced concrete structures increased drastically. After the revision of Japanese provisions in 2000 [29,30], the use of synthetic design ground motions became mandatory whose velocity spectra have the constant value of 80 cm/sec in the long-period range. The response spectra of the design ground motions were also defined at the engineering bedrock, and the amplification in soil should be considered at each site appropriately. Although consideration of the P-Delta effect is recommended in the current Japanese provisions, this is not mandatory. Many high-rise buildings are therefore still designed without considering the P-Delta effect.

III. SMRF Models

In this section, we design 9 high-rise SMRF models in accordance with the Japanese design practice in the 1980s. The main conditions for designing the SMRF models can be summarized as follows. More details of the design of the SMRF models can be found in reference [34]. Assuming regular SMRFs, we analyze plane frame models shown in Figure 1. The numbers of the stories of the models are 20, 30, or 40, while the number of spans is fixed to 3 as shown in Fig. 1(a). The floor supported by the plane frame is shown by the gray area in Fig. 1(b). The floor load is 7.85kN/m^2. All the columns have box sections and all the beams have H-shape sections. The nominal yield stress of these members is 325MPa. Column strengths are designed to be more than 1.5 times of the beam strengths based on the strong-column-weak-beam concept. Following the simplifying assumptions often applied in Japanese design practice [28,35], the following assumptions are made in this paper: (1) Composite actions between beams and concrete slabs are considered by doubling the elastic bending stiffness of the beams, while the increase of the strength is neglected. (2) To reflect the variation of yield stresses, 1.1 times of their nominal values are used. (3) Material nonlinearity is considered using a plastic hinge model with bi-linear kinematic hardening whose hardening parameter is 0.01. (4) None of local buckling, lateral buckling, and fracture takes place.

In the Japanese provision, the design base shear coefficient C_B is obtained as C_B=ZR_tC_0, where Z represents the seismicity of the region, R_t reflects the type of the soil, and C_0 is the standard shear coefficient. The minimum requirement of C_0 is 0.2. In this paper, we assume Z=1 and the type 2 soil. For the type 2 soil, R_t is obtained as R_t=0.96/T when the natural period T is equal to or greater than 1.2 sec. The Japanese provision provides an estimate equation of T (sec) as T=0.03H, where H (m) is the height of the building. In Japan, buildings taller than 60 m are usually designed by increasing C_0. To reflect this, we design the SMRF models by changing C_0 to be 0.2, 0.3, or 0.4, as a parameter to control strength of the SMRF models. To these design base shear values, allowable stress design is performed with the check of the maximum interstory drift ratio $\theta_{\max}$<1/200. The check of the ultimate strength is also required in the Japanese provision. For regular SMRFs, C_U/C_B >1.25 is required, where C_U is the base shear coefficient at the ultimate strength.

TABLE I summarizes the values that represent the structural properties of the 9 SMRF models. The model name “SMRF20-0.2” indicates that N=20 and C_0=0.2, where N is the number of stories. Note that the values of C_B in SMRF30-0.2 and SMRF40-0.2 models are both less than the minimum recommendation of 0.05 [31]. These models were, however, included for comparison purposes to other models. The ultimate strength base shear coefficient C_U was obtained by pushover analysis as the base shear coefficient at the maximum interstory drift ratio of 1/75 because no clear limit point was observed. The first-mode elastic period T_1 was obtained by the eigenvalue analysis. The ratio of C_U to C_B and that of T_1 to H are also shown in TABLE I. It is worth noting that the values of C_U and T_1 of SMRF20-0.2, one of the weakest and most flexible models, are close to those of the 20-story Japanese model (C_U=0.15 and T_1=3.04s) designed by Hall [10]. In TABLE I, Θ is the stability coefficient, which is defined as the ratio of the absolute value of the negative story stiffness caused by the P-Delta effect to the elastic story stiffness. The hardening parameter α is defined as the ratio of the postyield story stiffness at the ultimate strength to the initial elastic story stiffness. The subscripts NPD and PD indicate that the hardening parameters are obtained by the pushover analysis neglecting and considering the P-Delta effect, respectively. One can observe that α_{PD} decreases as the design base shear decreases or the SMRF model becomes taller, while the variation in α_{NPD} is relatively small.

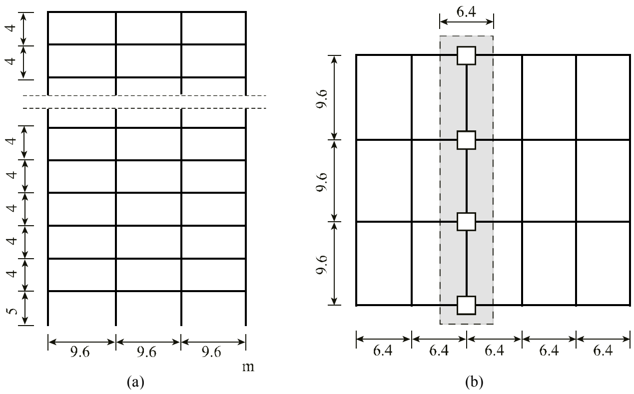

Fig. 1 The SMRF model: (a) elevation and (b) plan

TABLE I
REPRESENTATIVE VALUES FOR DESIGN OF SMRF MODELS.

SMRF	20-0.2	20-0.3	20-0.4	30-0.2	30-0.3	30-0.4	40-0.2	40-0.3	40-0.4
N	20	20	20	30	30	30	40	40	40
C_0	0.2	0.3	0.4	0.2	0.3	0.4	0.2	0.3	0.4
C_B	0.08	0.12	0.16	0.04	0.08	0.12	0.03	0.06	0.09
C_U	0.15	0.20	0.27	0.09	0.13	0.18	0.08	0.11	0.15
C_U/C_B	1.92	1.68	1.66	2.25	1.63	1.50	2.67	1.83	1.67
T_1 (sec)	3.06	2.56	2.18	4.59	3.78	3.29	5.95	4.90	4.23
H (m)	81	81	81	121	121	121	161	161	161
T_1 (sec)/H (m)	0.038	0.032	0.027	0.038	0.031	0.027	0.037	0.030	0.026
Θ (%)	4.48	3.24	2.46	7.90	4.15	2.83	8.53	4.33	2.91
α_{NPD} (%)	1.65	1.77	1.87	1.26	1.38	1.45	1.31	1.58	1.42
α_{PD} (%)	−1.49	−0.60	0.08	−2.55	−1.38	−0.76	−2.90	−0.82	−0.73

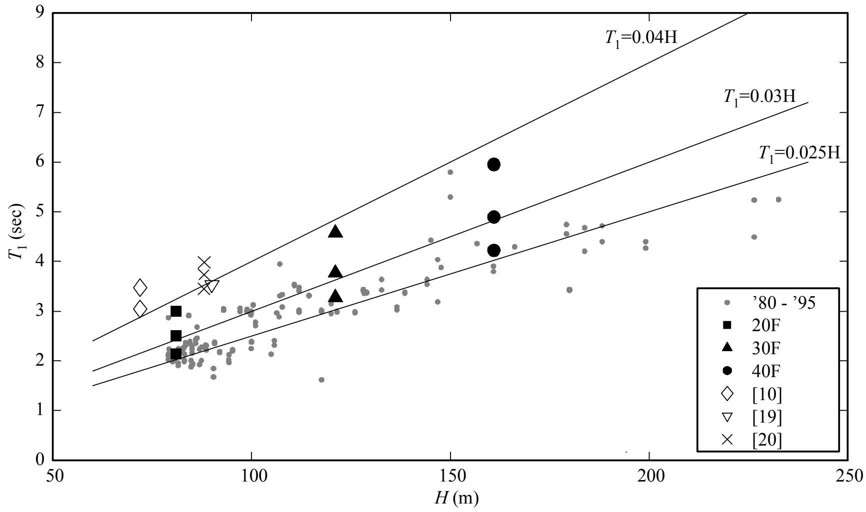

Fig. 2 Relationships between the building height H (m) and the first-mode elastic period T_1 (sec)

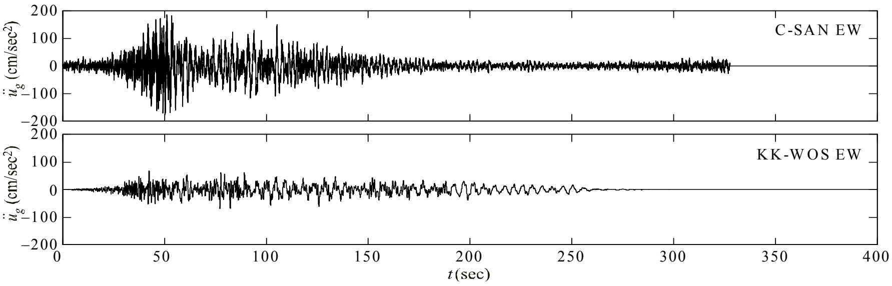

Fig. 3 Time-histories of C-SAN EW and KK-WOS ground motions.

Fig. 2 shows the relationships between the first-mode elastic period T_1 and the building height H, where the gray circles indicate those of the buildings whose design was reviewed by the BCJ during the period from 1980 to 1995. The black marks indicate those of the SMRF models considered in this paper. For reference, 3 lines are plotted in Fig. 2 indicating T_1=0.025H, T_1=0.03H, and T_1=0.04H. In Japan, the relationship T_1=0.025H is often used to estimate T_1 of steel buildings when the building is taller than 60 m. From Fig. 2, the following observations can be made: (1) The equation of T_1=0.04H approximates the upper bound of T_1 of existing high-rise buildings designed during this period in Japan. Most buildings satisfy T_1<0.03H. And, the equation of T_1=0.025H provides a good estimate. (2) The building models in references [10,19,20] that addressed dynamic instability of high-rise SMRF models lie in or close to the region defined by T_1>0.04H. This suggests that these SMRF models are much more flexible, and probably much weaker, than the average high-rise buildings constructed in this period in Japan.

As Level 2 ground motions, El Centro 1940 NS, Taft 1952 EW, and Hachinohe 1968 NS are scaled so that their PGVs are equal to 50 cm/sec. To these Level 2 ground motions, the following conditions are checked in this paper: (1) The maximum interstory drift ratio θ_{max} is less than 1/100. (2) The ductility factor is less than 4 in each member. (3) No yielding is allowed in columns except the column bases of the first story.

IV. Dynamic Instability

Architectural Institute of Japan (AIJ) organized a committee to study the vulnerability of long-period structures to long-period ground motions [1]. In the study, a number of synthetic long-period ground motions were generated. And the responses of many high-rise building models were analyzed using the synthetic long-period ground motions. Through the results, two long-period ground motions, named C-SAN EW and KK-WOS EW, were identified to have the largest influences on the response of high-rise buildings. In this section, we examine the response of the SMRF models presented in the last section to these two long-period ground motions. C-SAN EW and KK-WOS EW were predicted to occur in Nagoya and Osaka, respectively, caused by hypothetical Tonankai and Nankai earthquakes. The probability of occurrence is once in 100 years, which is relatively high. Fig. 3 shows the time histories of these long-period ground motions, where $\ddot{u}_g$ and t indicate the ground acceleration and time, respectively. Fig. 4 illustrates the response velocity spectra of the ground motions, where S_v and T indicate the response velocity and natural period,

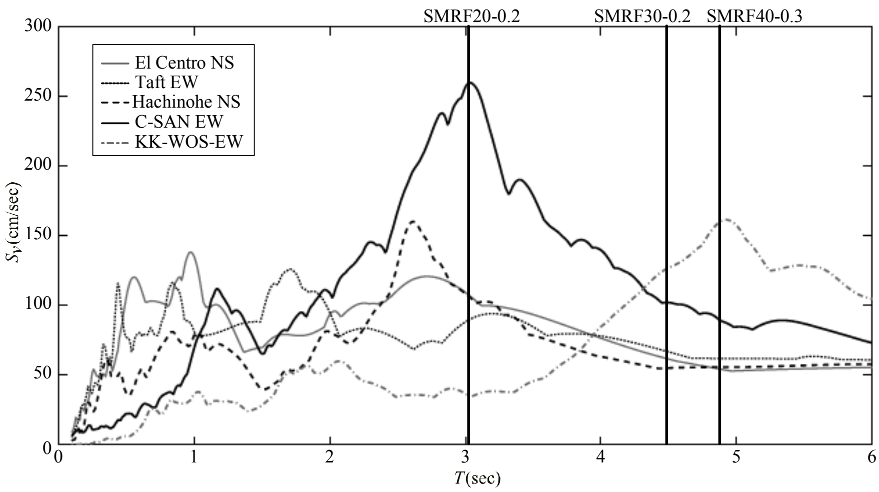

Fig. 4 Velocity response spectra of C-SAN EW, KK-WOS EW, and Level 2 ground motions (damping ratio=5%).

TABLE II
REPRESENTATIVE VALUES OF GROUND MOTIONS

ground motion	PGA (cm/sec^2)	PGV (cm/sec)	T_D (sec)	T_P (sec)	$S_v(T_P)$ (cm/sec)	earthquake	site	probability of occurrence
C-SAN EW	185.9	50.5	119.9	3.04	260.0	Tonankai (hypothetical)	Sannomaru (Nagoya)	once/100 years
KK-WOS EW	69.3	24.8	179.4	4.92	161.6	Nankai (hypothetical)	West Osaka (Osaka)	once/100 years
El Centro NS (scaled, Level 2)	509.0	50.0	53.7	0.99	138.2	Imperial Valley (1940)	El Centro	
Taft EW (scaled, Level 2)	496.9	50.0	54.4	1.72	126.3	Kern County (1952)	Taft	
Hachinohe NS (scaled, Level 2)	333.7	50.0	35.99	2.63	160.3	Tokachioki (1968)	Hachinohe	

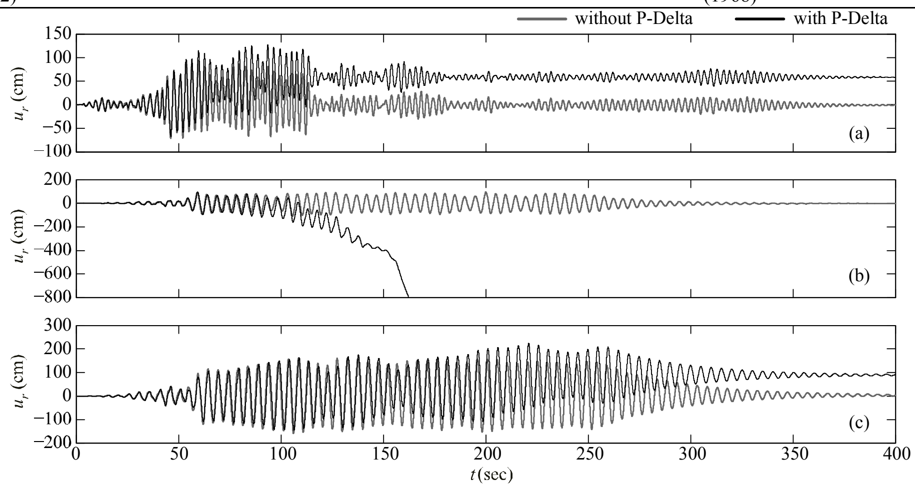

Fig. 5 Time histories of roof displacement: (a) SMRF20-0.2, C-SAN EW, (b) SMRF30-0.2, KK-WOS, and (c) SMRF40-0.3, KK-WOS

respectively. For comparison purposes, the response velocity spectra of the Level 2 ground motions are also shown in Fig. 4. TABLE II summarizes the representative values of the long-period ground motions. In TABLE II, T_D denotes the duration of the ground motions, and T_P and $S_v(T_P)$ indicate the dominant period and the peak value of the response velocity spectra of the ground motions. Note that, for the Level 2 ground motions, T_P was determined in the range where the natural period is larger than 1.5 sec. We performed nonlinear time history analyses of the 9 SMRF models using the frame models wherein both material and geometric nonlinearities [36] are considered. Material nonlinearity is considered using the generalized plastic hinges located at the both ends of a member. Geometric nonlinearity is considered by the geometric stiffness matrix and the moving

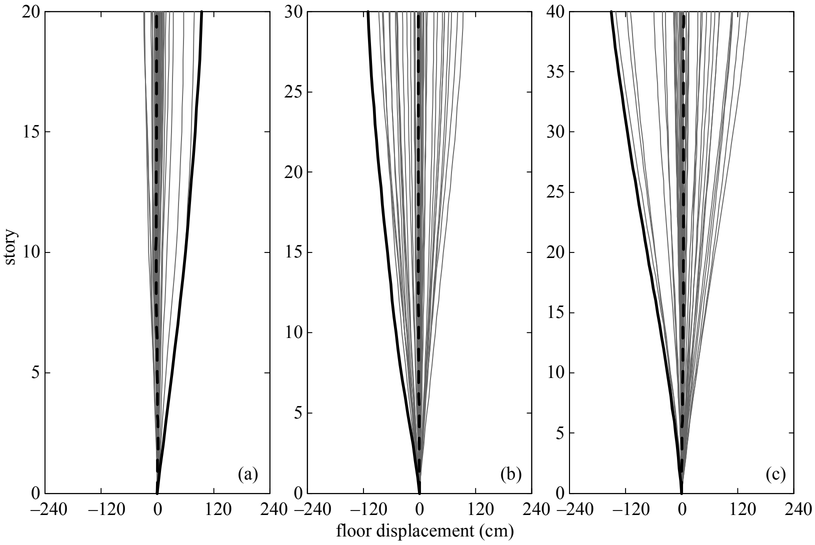

Fig. 6 Floor displacement neglecting the P-Delta effect: (a) SMRF20-0.2, C-SAN EW, (b) SMRF30-0.2, KK-WOS, and (c) SMRF40-0.3, KK-WOS.

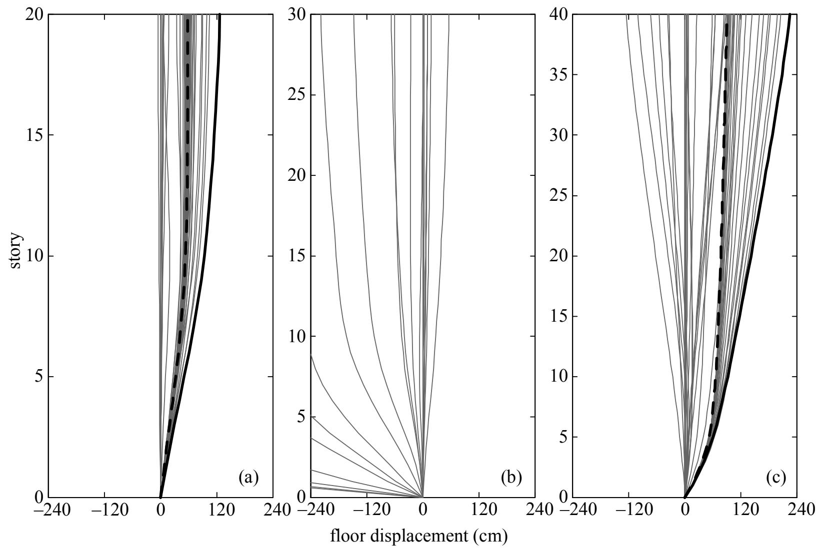

Fig. 7 Floor displacement considering the P-Delta effect: (a) SMRF20-0.2, C-SAN EW, (b) SMRF30-0.2, KK-WOS, and (c) SMRF40-0.3, KK-WOS.

coordinate system. Among the 9 SMRF models, drifting took place in the response of SMRF20-0.2 to C-SAN EW, and the responses of MRF30-0.2 and MRF40-0.3 to KK-WOS EW. The natural periods of these models are also shown by the bold vertical lines in Fig. 4. One can see from Fig. 4 that drifting took place when the natural periods of the SMRF models are close to the dominant periods of the long-period ground motions."

Figs. 5 to 7 illustrate the drifting responses. Fig. 5 plots the time histories of the roof displacement u_r, where black and gray lines indicate the responses when the P-Delta effect is considered and neglected, respectively. In Figs. 6 and 7, gray lines illustrate the floor displacement at the interval of 10 seconds, and the solid and dotted bold black lines indicate the maximum and the residual displacements, respectively. In Fig. 7(b), total collapse took place. Concentration of deformations predominantly in lower stories can be clearly seen in Fig. 7. It is worth noting that this type of collapse was also observed experimentally by Wada et al. [38], where a 12 story miniature SMRF model was subjected not only to horizontal base motions but also to high gravity load using a centrifuge machine. Figs. 5-7 suggest that consideration of the P-Delta effect is essential to capture this type of dynamic instability.

V. Conclusion

We have examined dynamic instability in 9 high-rise SMRF models subjected to 2 synthetic long-period ground motions, whose occurrence probability is once in 100 years. The 9 SMRF models were designed in accordance with the Japanese design practice in 1980s. In the design, the number of stories and the design base shear were changed as key parameters. The 2 synthetic long-period ground motions were supposed to take place in Nagoya and Osaka induced respectively by hypothetical Tonankai and Nankai earthquakes, whose occurrence probability in the next few decades are higher than 50 %. Drifting was observed when the natural periods of the high-rise SMRF models were close to the dominant periods of the long-period ground motions. Drifting appeared only in the lower stories of the SMRF models, and it was associated with the residual interstory drift ratio larger than 0.01. Among the cases where drifting took place, collapse was observed in the 30-story SMRF model. Such results demonstrate the potential risks in high-rise buildings to long-period ground motions whose probability of occurrence is relatively high. Although only non-structural components of high-rise buildings in Tokyo and Osaka were damaged in the Tohoku earthquake of March 11th, 2011, much more serious damage may take place if similar scale of a subduction earthquake occurs at the locations close to Tokyo, Nagoya, or Osaka. Hence proper assessment and seismic retrofit, if necessary, of high-rise buildings are urgent tasks

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